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U.S. Army Toxic and Hazardous Materials Agency

DEVELOPMENT OF A COMPUTERIZED PENETROMETER SYSTEM FOR HAZARDOUS WASTE SITE SOILS INVESTIGATIONS

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An instrumented cone penetrometer system has been developed for use in the investigations of soils at hazardous waste sites. The penetrometer is based on designs used for foundation investigations and includes instrumentation to measure tip penetration resistance, sleeve friction and electrical resistivity. The unit includes a computer-based data acquisition system that records input from the sensors and calculates indices related to the soil type. The resistivity module can detect the presence of soil contaminants that change the electrical properties of the soil. Field trials conducted with the penetrometer at Louisiana Army Ammunition Plant showed the penetrometer could be used to trace soil types and determine the extent of soil contamination at a closed waste-water evaporation pond. The penetrometer results were confirmed using standard soil sampling and laboratory analysis techniques. The penetrometer produced no cuttings that required disposal, and the unit could be rapidly decontaminated. Grouting equipment built into the penetrometer allowed the

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holes to be sealed as the penetrometer was withdrawn. A cost analysis prepared for a hypothetical soil investigation combining the penetrometer with conventional drilling indicates that the combined investigation would cost approximately 60 percent of the amount required if only drilling was employed. If the cost of laboratory chemical analyses for contaminants is considered, the potential cost reduction is much greater. The cost-effective strategy uses the penetrometer to characterize subsurface conditions and a minimum conventional drilling, sampling, and analysis program to confirm the findings. Placement of the confirmation borings and monitoring wells are guided by the penetrometer results.

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Installation Restoration General Environmental
Technology Development

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Final Technical Report

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TABLE OF CONTENTS

List of Figures

List of Figures
(continued)

List of Tables

1.0 EXECUTIVE SUMMARY

Cone penetrometer equipment is being developed at the US Army Engineer Waterways Experiment Station (WES) as a rapid and economical means for subsurface investigations at hazardous waste sites. The intent of this development is to provide reliable, cost-effective penetration equipment and techniques which will complement, but not replace, conventional drilling and sampling methods. In this role penetrometers could be used to fill in data between existing borings, or could be used in an exploratory mode to characterize subsurface conditions and to delineate potentially contaminated zones for confirmation by drilling, sampling, and analysis. In either case, significant savings in field and laboratory costs can be realized.

A prototype computerized cone penetration system has been developed and preliminary field testing has been carried out at the WES and the Louisiana Army Ammunition Plant (LAAP) near Minden, LA. This WES-developed system includes an industrial computer to provide real-time data acquisition and processing, a multi-sensor instrumented cone, a special umbilical cord to connect the cone to control units at the ground surface, and a chemical grout pressure injection system to seal the penetration hole as the penetrometer is withdrawn from the soil. Most of these capabilities are unique in terms of the current state of the art.

This report presents the overall design concept adopted for the penetrometer system, describes the system components and their functions, and outlines the procedures used in calibration and data acquisition.

Also presented are results of the proof-of-concept field testing conducted at the LAAP. Data from the field exercise are compared with results obtain by laboratory testing of soil samples recovered using conventional soil investigation techniques (drilling and split-spoon sampling).

The body of the prototype grouting cone penetrometer is 1.40 in. (35.7 mm) in diameter. The tip of the cone has a 60° conical point. A strain gage load cell fitted behind the point allows continuous measurement of the point penetration resistance of the soil as the cone is advanced. A friction sleeve fitted directly above the tip is also instrumented with strain gages to provide a measurement of the friction between the probe wall and surrounding soil. The cone tip area is 1.54 sq in. (10 sq cm) and the friction sleeve area is 23.25 sq in. (150 sq cm). The penetrometer is designed to provide a 750 tons/sq ft (0.732 kgf/sq cm) point penetration capacity. The hollow push rods for the probe are fabricated in 39-in. (1-m) sections and each section has a 0.6-in. (1.52-cm)-ID central hole. The probe is advanced with a hydraulic truck-mounted rig in one meter intervals; the typical advance rate is 0.4 to 0.8 in./sec (1 to 2 cm/sec).

A soil resistivity measurement module is built into a rod section that fits directly above the friction sleeve of the penetrometer. This module has four electrodes spaced 2 in. (5 cm) apart with insulating collars between them. Current flows between the outer electrodes, and the potential drop in the soil is measured between the center electrodes. By measuring the current and the voltage drop, the apparent resistivity of the soil can be calculated.

Measurements of soil resistivity are useful in determining soil type and can also be used to detect the presence of contaminants.

The penetrometer is equipped with two plastic grout tubes that are run through the center of the rod and end behind an ejectable (sacrificial) plug at the cone tip. After the push is concluded, the rods are raised a few inches and a two-component expansive chemical grout is forced into the grout tubes under pressure. The sacrificial tip is ejected and the hole is grouted as the cone is withdrawn. The grout system can use a variety of two-component chemical grouts including sodium silicate, acrylate, or polyurethane-based materials. The gelling of the chemical grouts occurs rapidly after the grout components mix assuring the probe hole is sealed. Grout injection pressures are adjusted to prevent the fluid grout from hydraulically fracturing surrounding soil. The grouting system has been tested and provides a complete and continuous plug of the penetrometer hole. No contaminant migration can occur after grouting.

The proof-of-concept field testing at LAAP was carried out at a site which had been used as an evaporation pond for acidic wastes neutralized prior to disposal. The wastes are reported to have involved both solids and liquids. On closure, the evaporation pits were filled with debris and additional solid wastes and partly filled in with soil.

Cone penetrometer pushes were made and data were recorded for eight locations at the site. After the cone pushes were completed a drill rig was brought onto the site and a sample hole was drilled within 3 ft (0.9 m) of each of four penetrometer holes. Continuous drive or split-spoon samples were collected in each sample hole. The split-spoon samples for three boreholes were divided longitudinally to prepare identical soil samples. One set of the duplicate soil samples was submitted for laboratory classification to determine soil type. The second set of duplicate samples was submitted for laboratory soil resistivity and soil pH measurement. The soil analysis techniques employed followed standard procedures.

This computerized penetrometer system measures strength properties (sleeve friction and penetration resistance) and uses these data to calculate the apparent soil classification, i.e., clay, sand, etc., of the insitu materials. The laboratory soil classification is based on the sieve analysis and Atterberg limits determinations made on soil from each 18-in. (0.46-m) sample interval. Correspondence between the two classification systems is not exact. Most major soil transitions noted by the cone measurements can be documented in the laboratory results. The laboratory data provides less detail than the cone. Cemented near-surface layers were not correctly identified by the penetrometer classification algorithm.

At all three borings, the penetrometer data showed a trend of decreasing resistivity with increasing depth. The laboratory slurry pH showed an increase with depth in all three sample borings indicating the probable effects of alkaline waste disposal.

In all three borings, the water table stood at 9 to 12 ft (3 to 4 m) below the surface, but the saturated zone does not appear as the dominant feature in the

resistivity measurements. Comparison of the slurry resistivity data (made on a constant weight of sample and a set volume of liquid) and the in-situ cone resistivity (where the degree of saturation varied) indicated the cone resistivity was also responding to increased concentrations of dissolved salts.

The field test demonstrated the following:

- Soil strength measurements made with the penetrometer can be used to infer major changes in soil type in the subsurface.

- Soil resistivity measurements made in situ show trends that reflect the laboratory-based soil resistivity measurements.

- Laboratory soil pH measurements generally increase with decreasing soil resistivity indicating alkaline soil contamination is being tracked.

- The instrumented cone penetrometer provides a rapid method for locating soil zones of anomalous resistivity and of locating major changes in soil type rapidly with the minimum problems of instrument decontamination. No samples are needed and therefore sample disposal and site cleanup are not required.

This penetrometer system offers unique advantages in investigation of waste sites in that:

- A full set-up and push to 50 ft (17 m) can typically be completed in less than one hour.

- The data are available on a real-time basis as a computer display, and a printed record is available before the push rig is moved.

- The continuous records produced by the probe are greatly superior to boring data for stratigraphic correlation across the site.

- The decontamination of the cone is accomplished by washing or wiping down the rod as it is retracted.

- No samples are taken and no cuttings are produced, so exposure of the crew is minimal.

- Contaminated zones which have altered electrical properties can be detected using resistivity measurements.

- Any cavities or obstacles in the subsurface are immediately identifiable from changes in cone penetration resistance measurements.

- The hole can be grouted closed before the push rig is moved to another location.

The penetrometer can be adapted to carry other specialized sensors and to take high-quality liquid or gas samples. Computerized data processing and a

virtual real-time display of results make the probe a versatile tool for rapidly mapping contaminant plumes in the soil.

2.0 INTRODUCTION

2.1 Background

The Cone Penetrometer Test (CPT) was originally developed in Europe as a rapid and cost-effective means of determining soil layering and soil strength parameters. In Belgium and Holland, where soft soils predominate, the CPT has been used for 50 years as the most popular means of subsurface exploration to characterize soil conditions and to provide the soil strength data needed for foundation design (Begeman, 1974). Much of this testing was done with simple mechanical cones, however, the CPT has recently benefited from the development of instrumented electronic cones of improved accuracy and capacity. As a result of this development, the CPT has increased in popularity in the United States and has been recognized as a standard test method under American Society of Testing and Materials CPT Standard D-3449. Similar CPT standards have been established in many other countries worldwide including France, Sweden, Belgium, Holland, West Germany, England, Japan, Canada, and Mexico, to cite only a few examples.

The CPT method utilizes a hydraulically-powered push apparatus, usually truck-mounted, to force an instrumented cone penetrometer into the soil media as shown in Figure 2-1. Standard electronic cones have a 60^{o} cone tip, of 1.4-in. (35.7-mm) diameter, and typically include two load cells to simultaneously measure tip penetration resistance and skin friction as the cone is advanced. These measurements provide a continuous record of soil resistance to penetration which can be used to characterize the soil media in detail.

Recent advances in instrumentation technology have prompted the design and development of specialized multi-sensor cone penetrometers for a variety of new applications, including hazardous waste site (HWS) assessments. In HWS assessments, it is necessary to detect, delineate, and identify contaminants and to characterize subsurface conditions in sufficient detail to make meaningful predictions of contaminant migration over time. Current practice is to rely on a program of exploratory drilling, sampling, and laboratory analysis of soil and groundwater samples to obtain the needed information on subsurface conditions. Drilling/sampling/sample analysis programs are expensive and, for this reason, development of a less costly approach to subsurface exploration is attractive.

Incorporating hazardous waste sensors in cone penetrometers could provide a cost-effective means of satisfying the soils exploration requirements in HWS assessments. Cone penetrometers have also been used to obtain soil, groundwater, and gas samples in situ, and this technology could be adapted to both the exploration and long-term monitoring phases of HWS assessments. For these reasons the cone penetrometer is considered to be a leading candidate for further development to suit HWS assessment needs. Although conventional drilling/sampling/sample analysis programs cannot be eliminated, the prospect is that effective use of cone penetrometers could permit substantial reductions in the scope of such programs and costs of HWS assessments.

2.2 Purpose

The purpose of this report is to document the development of a computerized cone penetrometer system for HWS assessments. This report describes the development process, the prototype equipment, the successful proof-of-concept field testing, and the current status of the program to develop innovative technology for future deployment.

2.3 Objectives

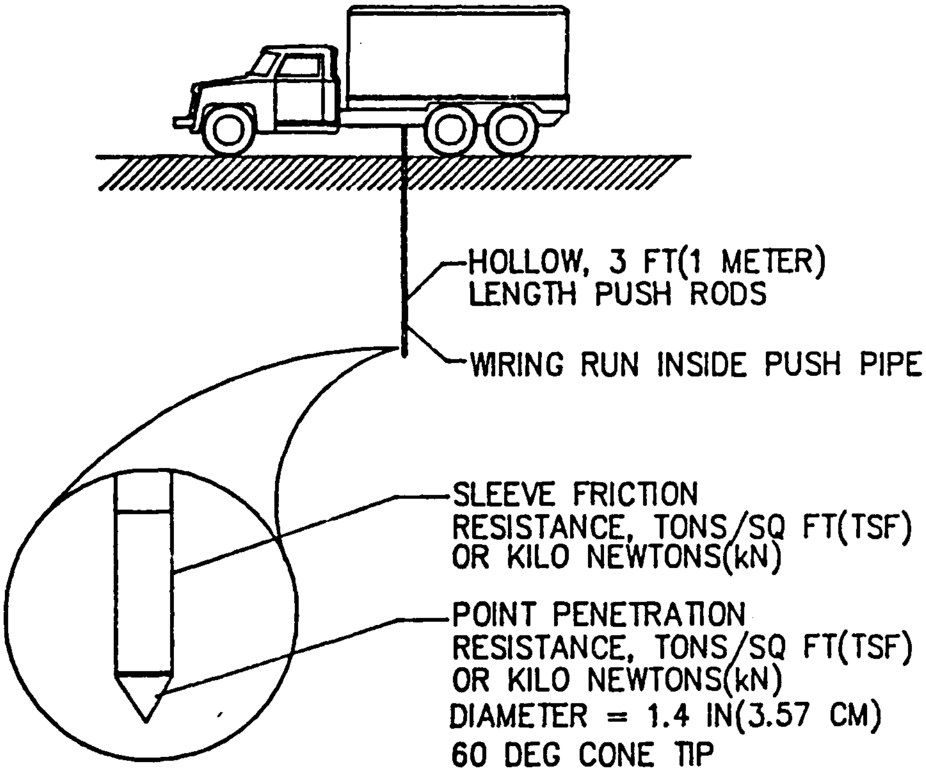
The first objective of this program is to develop a rapid and cost-effective means to characterize soil conditions and to detect/delineate/identify contaminants in situ. This objective involves substantially reducing the time and costs of site assessments which are now carried out using conventional drilling, sampling, and laboratory testing. The strategy will be to use cone penetrometers in the exploratory role, in order to characterize the subsurface soil conditions in detail, and to determine optimum locations for verification borings. Only a few borings would then be required to verify findings with physical samples for laboratory testing. An additional objective is to use the penetrometer data to optimize monitoring well placement and for design of remedial treatment systems.

3.0 DESIGN CONSIDERATIONS

A variety of cone penetrometer push devices are available including trailer-mounted, skid-mounted, and truck-mounted units. The most sophisticated of these are heavy-duty (20+ ton) trucks which feature air-conditioned work spaces and some limited form of computer-assisted data acquisition. Most equipment of this kind uses standardized 3-ft (0.9-m)-length push rods and a chuck to clamp on the rods for pushing or extracting a cone. The push rods have a center hole which permits their use with instrumented cones, which require a connecting cable(s) between the cone and truck-mounted instrumentation.

The WES design philosophy was to make maximum use of standard practices and equipment where possible, so as to minimize development time and cost. This approach was also used in evaluating the state of the art in contaminant sensor technology, i.e., first priority for development was given to sensors using proven methodology which could readily be adapted to the penetrometer equipment. The development of more innovative (hence higher risk) technology could proceed concurrently, but with the expectation that this development process would require more time to complete. Staging development in this way allows early fielding of a competent system, provides the means to optimize system performance through field testing, and permits incorporation of advanced technology as it is developed. The following general design concepts were evolved:

- The basic electric cone unit would conform to ASTM Standard D-3449. This would facilitate direct comparisons with the world-wide data base obtained using conventional electric cones.



TYPICAL CONFIGURATION

MOUNTED ON ALL WHEEL DRIVE TRUCK
HYDRAULICALLY POWERED PUSH APPARATUS
WEIGHT APPROXIMATELY 20 TONS (18.2 METRIC TONS)
COMPUTER AIDED DATA ACQUISITION
CONTINUOUS MEASUREMENTS OF PENETRATION RESISTANCE
ENCLOSED AIR-CONDITIONED WORKSPACE

Figure 2-1. Components of a typical cone penetrometer test system.

- Penetrometer hardware would be designed for compatibility with standard equipment. This would permit using the prototype penetrometer system with standard length pushrods and commonly available push devices.

- The prototype system would be designed to use computerized data acquisition and on-site data processing. This would allow virtual real-time presentation of results in the field and would handle the extensive data base generated in production testing.

- Sensor modules would be developed to detect contaminants insitu. Modular design would offer flexibility and provide the means to incorporate advanced technology without radical design changes. A soil resistivity measurement module would be accorded first priority for development. Resistivity techniques have been widely used in prospecting for hydrocarbons (Schlumberger Corp., 1950) and have also been applied to geotechnical problems (Guyod, 1969; Cooper, Koester, and Franklin, 1982). This proven methodology could be readily adapted to the cone penetrometer. More innovative sensor technology, such as fiber optic fluorimetry, would be developed concurrently.

- A grouting system would be developed to seal the penetrometer hole as the penetrometer is withdrawn. This would prevent any contaminant migration which might otherwise occur, and would best satisfy regulatory requirements that the holes be sealed.

A block diagram of the prototype system design is shown in Figure 3-1. The prototype equipment and details of its design and application are described in the following sections.

4.0 DESCRIPTION OF EQUIPMENT

4.1 Grouting Cone

The principal design criteria adopted for the grouting cone were as follows:

- To provide the means to achieve a continuous, effective, and chemically resistant plug in the hole as the penetrometer is withdrawn from the soil to prevent contaminant movement.

- To preserve the standard electric cone geometry, i.e., 60^{o} point, 1.4-in. (3.56-cm) OD, 1.54-in.^{2} (9.9-cm^2) tip area, and 23.25-in.^{2} (150-cm^2) friction sleeve area.

- To provide for modular additions to the basic design, for example, resistivity, fluorimetric, or other contaminant sensors as these are developed.

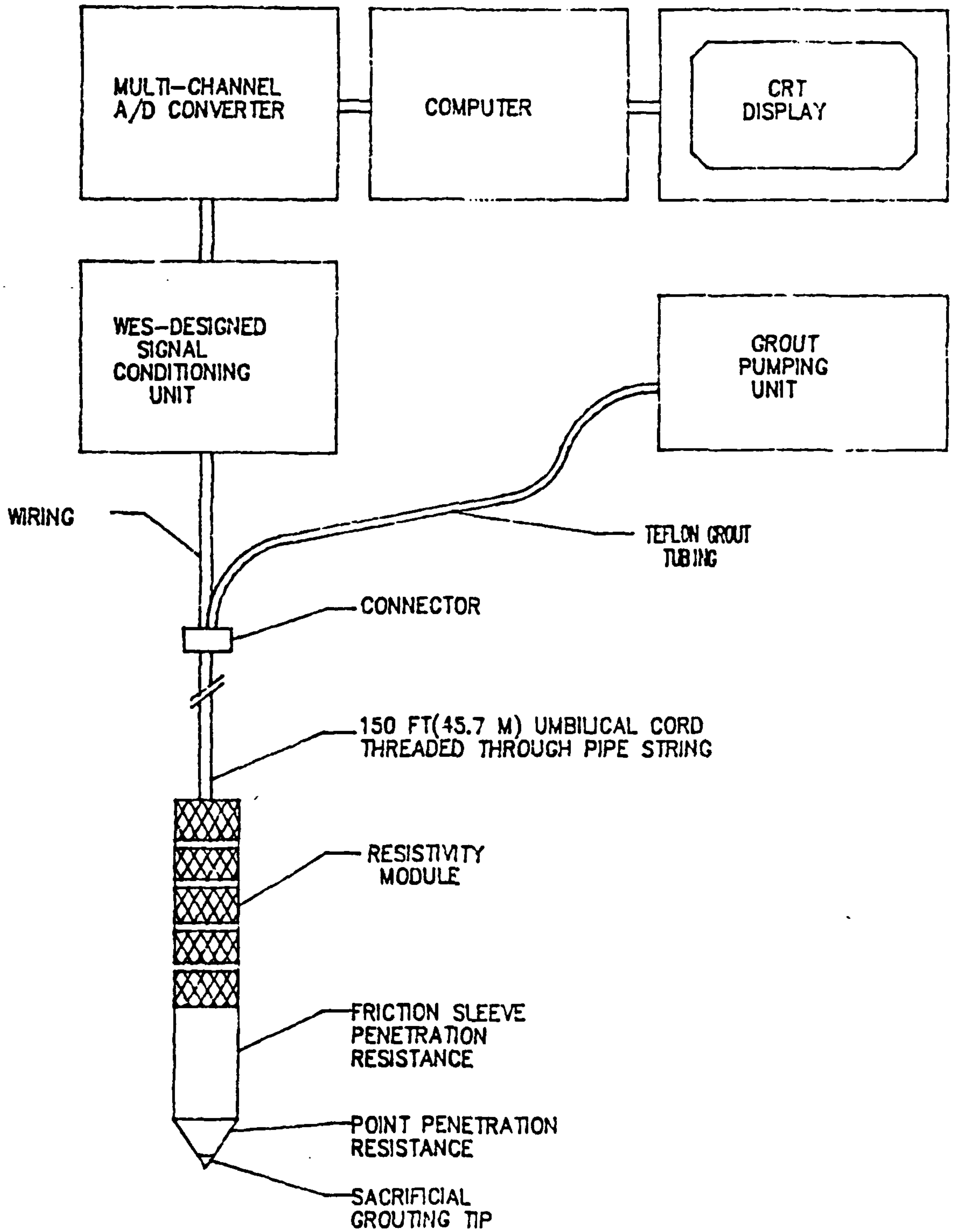


Figure 3-1. Concept for a computerized penetrometer system.

- To provide the structural strength necessary to develop at least 750 tons-per-square-foot (tsf) (732 kgf/cm^2) point penetration capability and concomitant friction sleeve capability, i.e., not less than 10 tsf (9.8 kgf/cm^2).

A chemical grouting system was selected for several reasons as being the most promising approach to the first objective. Some of the desirable grout attributes sought were fast setting time, expansive behavior, low viscosity, low permeability, resistance to chemical attack, and absence of toxicity. The grout could not be much more viscous than water (1 centipoise) or contain particles larger than about 30 microns because of the small tubes and orifices which would have to be used to pump the grout from the surface down to the penetrometer tip. This physical limitation is imposed by the 0.60-in. (1.52-cm)-ID hole in the center of the standard penetrometer push rods, which must accommodate not only the grout tubes but the uphole instrumentation cable as well. Chemical grouts appeared to offer more advantages than cement-based grouts in terms of low viscosity, small particle size, chemical resistance and setting time, and offered the additional advantage of being resilient enough to accommodate considerable soil movements without cracking or rupturing.

The second objective was to preserve the ASTM standard electric cone exterior geometry so as to facilitate direct comparisons with the worldwide data base obtained using conventional electric cones.

The third objective, that of providing for modular additions, is important because recent developments in sensor technology offer the prospect of detecting contaminants via resistivity contrasts, or ultraviolet light-induced fluorescence (using a fiber optic link from the surface to the penetrometer). A design concept has been developed at WES for UV fluorescence/reflectometry. In the time since WES first proposed UV fluorescence (1983) as a promising pollution sensing method, other investigations (Chydyk, Carrabba, and Kenny, 1985) have proven that this method can remotely detect a number of ground water contaminants at very low concentrations, i.e., in the part per billion range. This important development enhances the prospects for successful development of a fluorimetric penetrometer probe.

The final objective was to provide sufficient structural strength so that the penetrometer could penetrate dense soil layers without suffering significant damage. This posed some problems because considerable internal space had to be dedicated to the grouting system while preserving the standard electric cone exterior geometry. A sectional view of the prototype grouting cone is shown in Figure 4-1. The penetrometer body was machined from 1141 series cold-rolled steel, round stock which is relatively free machining and has a yield point typically in excess of 50,000 $lb/in.^2$ (3515 kg/cm^2) without heat treatment. The remaining components of the cone were fabricated from 300 series stainless steel, primarily for its corrosion resistance. The point load cell is located as shown in Figure 4-1 and is loaded in compression when the cone tip is advanced. The friction sleeve load cell is in the form of a hollow cylinder which is split along its cylindrical axis and strain gaged on the inside surface of each half shell. This cell surrounds the tip load cell

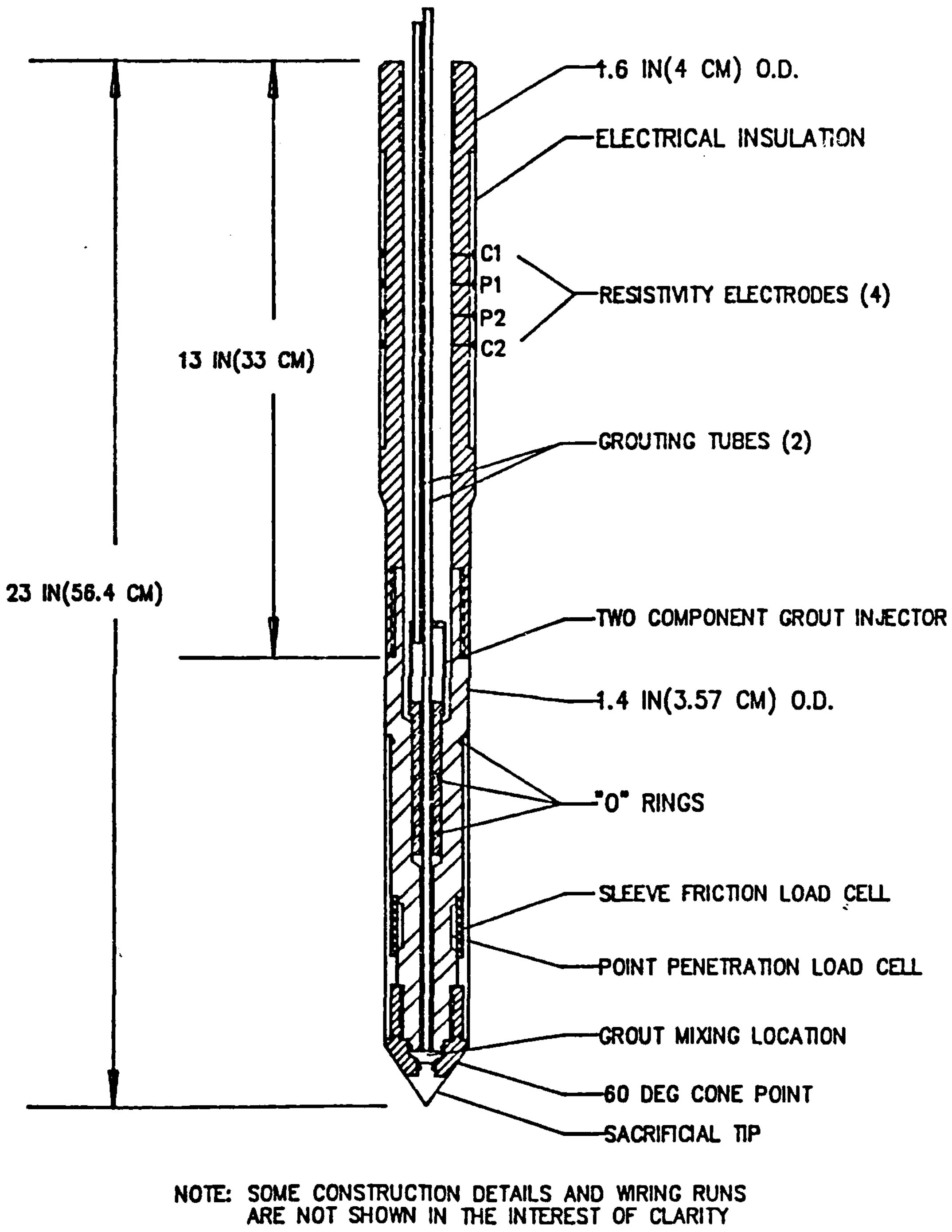


Figure 4-1. Cross-sectional view of the prototype multi-sensor cone.

and is also loaded in compression when soil friction acts on the friction sleeve which jackets the front of the probe. Thus, tip penetration resistance and sleeve friction measurements can be made independently and continuously. This kind of construction offers a significant advantage over so-called "subtractive" cones, whose construction requires that the tip resistance measurement be subtracted from a combined tip resistance and friction sleeve measurement in order to derive sleeve friction data. The calibration and subtraction processes inevitably cause some sacrifice in accuracy, and the resulting error can be large when sleeve friction forces are comparatively small, as in the case of soft clays. Errors of this kind are undesirable because sleeve friction measurements play an important role in deducing soil type for site characterization purposes.

Using the load cell configuration discussed above the prototype grouting penetrometer is rated as follows:

Max. tip force = 16,000 lb = 750 tsf (732 kgf/cm^2) penetration resistance

Max. sleeve force = 4,000 lb = 12.4 tsf (12.1 kgf/cm^2) sleeve friction

Combined max. force = 20,000 lb = 930 tsf (908 kgf/cm^2)

These ratings are considered to be ample for most soil conditions (which typically require less than half the rated capacity) and the prototype is capable of enduring a 30 percent static overload without damage.

As shown in Figure 4-1, the chemical grouting injector is housed within the penetrometer body and communicates with the tip via a 0.27-in. (70-mm)-diam hole along the probe centerline. At the extremity of the 60^o cone tip is a small, "O" ring-sealed, sacrificial steel plug which is blown out by fluid pressure when grouting commences. Approximately 5 to 10 psi (34 to 68 kPa) applied pressure is sufficient to dislodge the plug. The remainder of the downhole grouting system consists of a tube assembly which conveys the chemical grout components through the probe body to the tip where mixing occurs just behind the seat for the sacrificial plug. Two 0.25-in. (64-mm)-ID teflon tubes are used to pump the urethane prepolymer and water (or other two-component grout materials) from the ground surface through the push rods into the grout injector in the probe. This configuration insures that fast setting two-component grouts, such as urethanes, can be used with little or no clogging of interior orifices because mixing occurs at the tip extremity rather than inside the probe. In any case, the injector assembly is designed to be easily removed for cleaning should clogging occur. According to design calculations the injector system and tubing should cause no more than a 20 psi (138 kPa) pressure drop using fluids with a viscosity close to that of water, but hydrostatic pressure existing at the depth of injection must also be overcome in order to dispense the grout mix. Assuming a 150-ft (49.2-m) maximum depth of penetration, and the water table at the ground surface, the hydrostatic pressure opposing injection would be about:

$$\frac{62.5 \text{ lb}}{\text{ft}^3} \times \frac{1 \text{ ft}^2}{144 \text{ in.}^2} \times 150 \text{ ft} = 65 \text{ psi } (448 \text{ kPa})$$

In most cases grouting pressures from 5 to 15 psi (35 to 105 kPa) above the existing hydrostatic pressure and friction losses should be ample, so the maximum pressure, P_{max}, required from the system should not exceed:

$$P_{max} = \text{Frictional losses} + \text{max. hydrostat} + \text{grouting pressure}$$

$$P_{max} = 20 \text{ psi} + 65 \text{ psi} + 15 \text{ psi} = 100 \text{ psi } (689 \text{ kPa})$$

This pressure level should not pose a problem because the tubing and other downhole components of the grouting system are service rated for 300 psi (2067 kPa). Means of monitoring the grout pressure and flow must also be provided at the ground surface because the volume of grout dispensed should be known to be at least equal to the hole volume, and the grout pressure should be controlled to prevent the phenomenon called "hydrofracturing". Hydrofracturing can occur when injection pressure is allowed to significantly exceed the minimum principal stress in the soil at the depth of injection; the soil layers then tend to lift and separate, or fracture or do both. It is prudent to use the least pressure needed to achieve the desired results. Under excessive grouting pressure a significant volume of grout could be lost to the voids created by hydrofracturing, with an attendant increase in time and costs. For an appreciation of the quantity of grout required one can assume a 150-ft (49.2-m) depth of penetration in a relatively impervious soil, for example a massive clay. The least volume of grout needed to completely fill the hole left after penetration is about:

$$V = \frac{\frac{D^2}{4} \times 150 \text{ ft} \times \frac{12 \text{ in.}}{\text{ft}}}{\frac{231.8 \text{ in.}^3}{\text{gal}}} \times \pi = 15.6 \text{ gal } (59.05 \text{ ℓ})$$

where D is the assumed hole diameter, about 1.6-in. (4-cm) OD.

In more pervious zones, such as poorly graded sands, the volume of chemical grout take will ordinarily be greater than the hole volume since the grout can flow into the sand layer. An accurate estimate of the maximum grout storage volume needed to satisfy all soil conditions for one 150-ft (49.2-m)-depth hole is difficult to formulate because of the many variables which can affect the results. These variables include grouting pressure and time applied, grout particle size, grout viscosity, net permeability of the formation(s), grout setting (gelling) time, etc. However, as a first approximation to the problem it can be assumed that for production testing the two-component grout storage containers should be no less than 50 gal (190 ℓ) total capacity, which will provide about three times the volume needed to completely fill a 150-ft (49.2-m) depth of 1.6-in. (4-cm)-diam penetrometer hole. No allowance is made in those calculations for grout expansion or the tendency of the hole to close due to lateral earth pressure as the penetrometer is withdrawn. These conditions would tend to reduce the total quantity of grout take, but an evaluation of all of the variables affecting grout take can best be done through field testing of candidate chemical grouts. The effectiveness of this grouting system is illustrated by a test performed at the WES using similar procedures in a loess soil. After grouting, a backhoe was used to excavate a trench to

expose the penetrometer hole. As shown in Figures 4-2 and 4-3, the grout formed a continuous and expansive plug of the penetrometer hole from the surface to the maximum depth of penetration.

4.2 Resistivity Measurement Module

A schematic of the soil resistivity measurement circuit is shown in Figure 4-4. The resistivity module utilizes four metal electrodes equally spaced in a Wenner array on an electrically insulated section of the resistivity module. The electrical insulation is a mixture of three parts by volume of carborundum particles and one part of high-strength epoxy. This mixture was cast in place on the outer surface of the resistivity module. The outer face of each metal electrode is flush with the surface of the insulation. The maximum diameter of the resistivity module is 1.6 in. (4 cm), as shown in Figure 4-1.

It should be noted that this resistivity circuitry requires two simultaneous measurements, and that a bipolar DC voltage is applied to excite the soil media. Bipolar DC excitation eliminates any undesirable effects of electrode polarization, which might otherwise occur. Two simultaneous electrical measurements are made in order to satisfy the general resistivity equation for a four-electrode in-line array in an infinite medium, which is:

$$\rho = 4\pi a k \frac{\Delta V}{I}$$

where

ρ = resistivity, ohm-ft or ohm-m

a = electrode spacing, ft or m

k = calibration constant which accounts for penetrometer geometry effects (dimensionless)

ΔV = voltage drop across the measurement electrodes (volts)

I = current in the excitation circuit (amperes)

The constant $4\pi ak$ is a fixed quantity for any given electrode geometry and can be obtained from calibration testing. One then need only measure the current, I , in the soil media (excitation circuit) and the voltage drop, ΔV , across the measurement electrodes, in order to calculate resistivity ρ .

Soil resistivity measurements have been proven useful in borehole logging to determine soil stratigraphy and soil type when used in conjunction with other measurements such as spontaneous potential (SP). Soil resistivity measurements are potentially useful for HWS investigations because the presence of some contaminants can significantly change the resistivity of the ground water and soil. For example, if the equivalent salinity of the ground water increases from 100 to 200 ppm, then the resistivity of the ground water is roughly halved, i.e., from about 155 ohm-ft (51.15 ohm-m) to 80 ohm-ft (26.4 ohm-m). In fact, the resistivity of an electrolyte (ground water), decreases roughly in proportion to the number of ions of any type present in

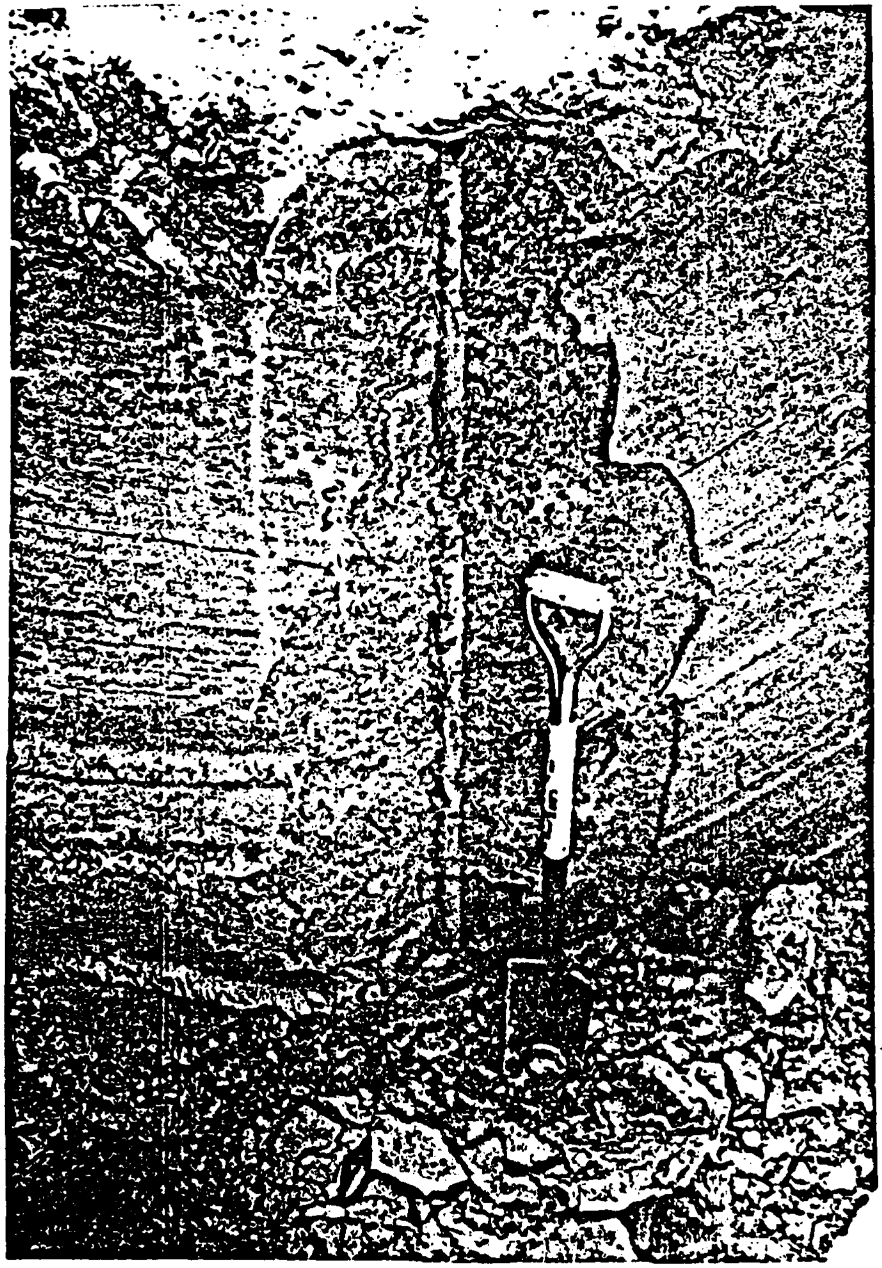


Figure 4-2. Excavated penetrometer hole showing complete sealing developed with urethane foam grout.

Figure 4-3. Top of the penetrometer hole showing the bulb formed by the expanding urethane foam grout.

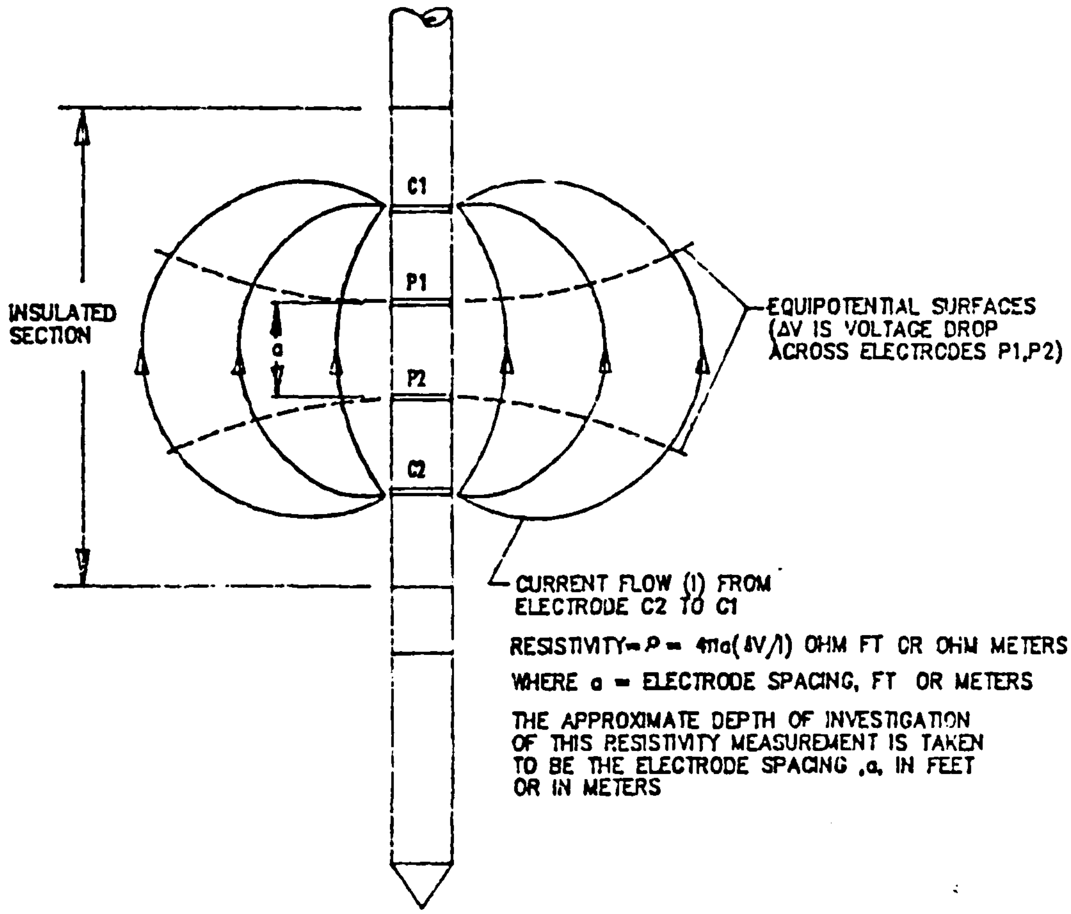


Figure 4-4. Schematic representation of Wenner electrode configuration in soil.

the solution (Schlumberger Corp., 1950, 1972) so, soil resistivity measurements can provide an index of contaminant concentration. Recent WES experience at munitions plants indicates that ground-water contamination can easily reach a thousand parts per million equivalent concentration. At these concentrations there can be a pronounced soil resistivity contrast between contaminated and uncontaminated zones, as shown in Figure 4-5. The degree of resistivity contrast will depend on the type and concentration of contaminants, and only a few contaminants (sulfate, salts, oil) have been evaluated in terms of the resistivity contrast they produce. However, the presence of either organic or inorganic wastes in groundwater should produce some degree of resistivity contrast and moderate to high concentrations of contaminants should be detectable with resistivity methods as evidenced by the data in Figure 4-5.

Resistivity and SP measurements are considered to be proven technology which can readily be adapted to HWS applications. Naturally occurring DC self potentials, or SP, are generated by electrochemical, electrokinesis, and telluric current phenomena. Of these, the electrochemical and electrokinesis phenomena typically produce the largest voltage magnitude (on the order of 1 volt, or less), and are readily measured using a precision high internal impedance voltmeter. The utility of the SP measurement is that it can aid in the determination of soil layering, and can also be used to detect ground water migration (Cooper, Koester, and Franklin, 1982) in favorable circumstances.

4.3 Data Acquisition System

A PC industrial microcomputer was selected for use in this project because of the wide range of devices available for it and because the manufacturer had "ruggedized" the unit to an appreciable extent. Other advantages included a relatively fast (8 MHz) internal clock speed, which is desirable in multichannel analog to digital (A/D) conversions and for virtual real time data processing, and the convenience of standard instrumentation rack mounting.

The computer was purchased with 512 Kbytes of random access memory (RAM), a 1.2-Mbyte floppy disk drive, and a 20-Mbyte hard disk so as to accommodate a substantial data base (approximately 100 penetrations to a 150-ft (45.7-m) depth before the hard disk memory storage is exhausted). Provisions were also made to incorporate a tape recording system and an uninterruptable power supply to prevent data loss in the event of a power failure during operation.

The computer was also fitted with an enhanced graphics package for additional flexibility in CRT (TV monitor) display, and a printer and plotter for producing hard copy. The system unit, monitor, and peripherals were all mounted in an instrument rack equipped with a storage drawer, writing surface, and lockable caster wheels, as shown in Figure 4-6.

The computer provided eight expansion slots for peripheral multifunction cards. One of these slots was used for a memory expansion card to add 512 Kbytes of RAM. This increased the total RAM capacity to 1 Mbyte, which was desirable for running various data acquisition programs.

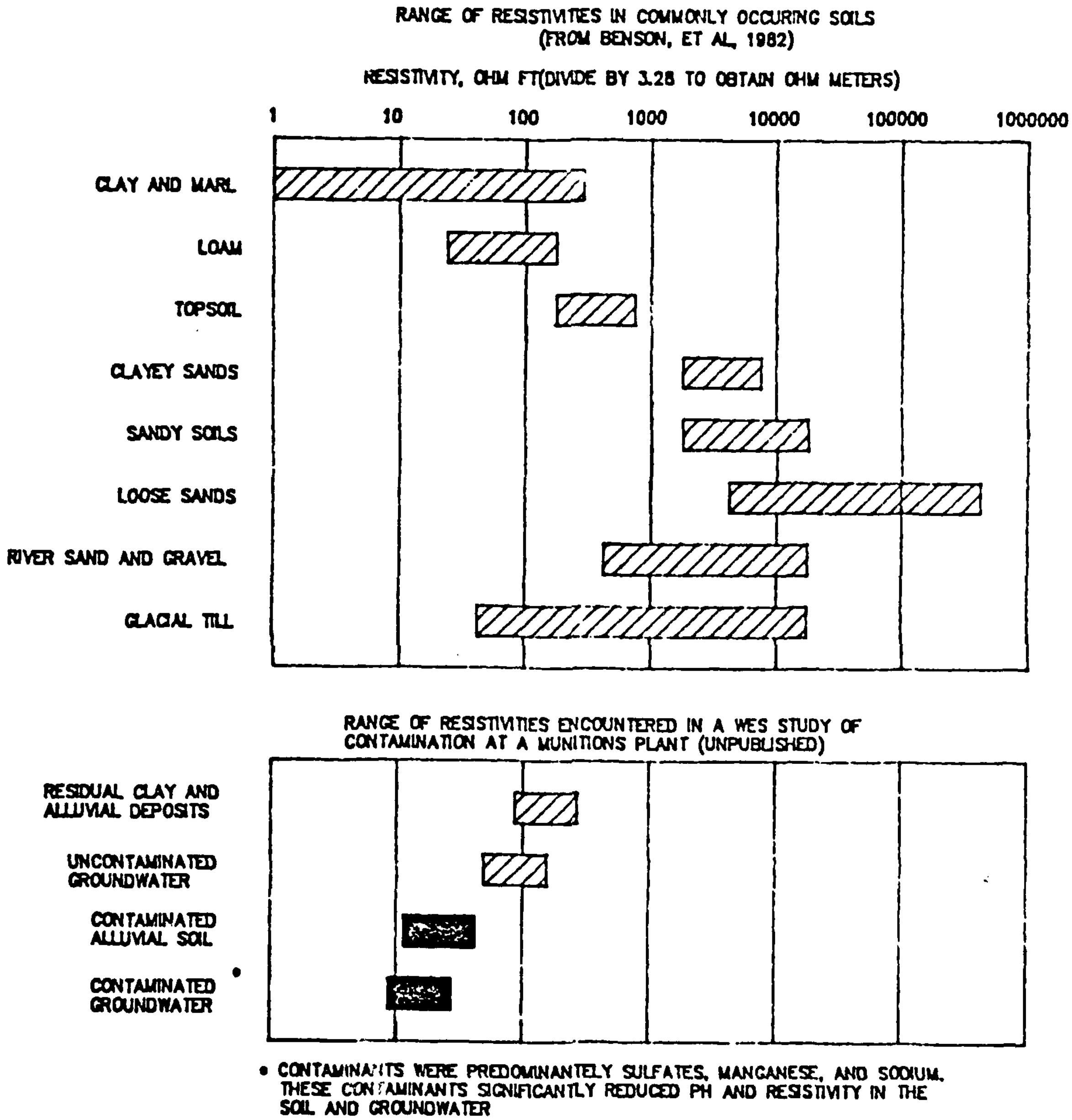


Figure 4-5. An example of the resistivity contrast which can occur when moderate to high concentrations of contaminants are present in the groundwater.

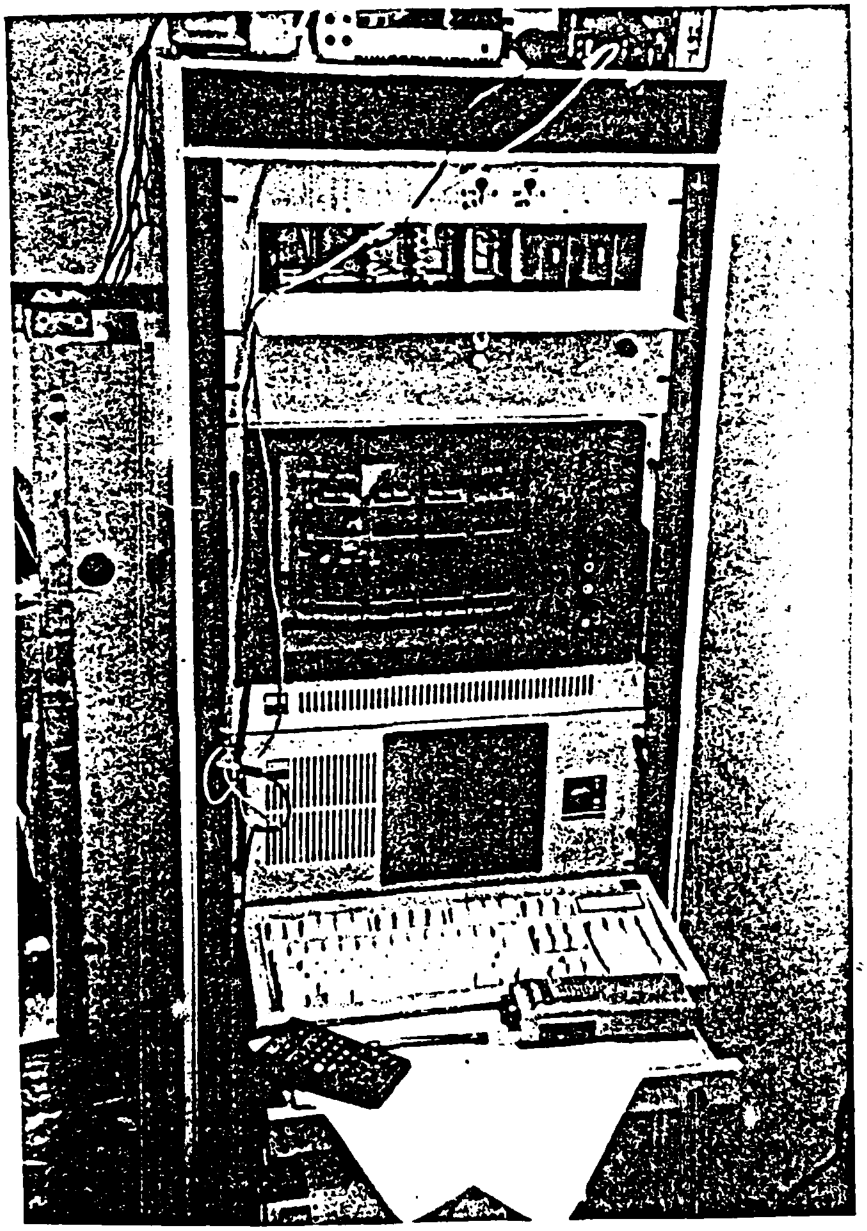


Figure 4-6. Console mounted computer and peripheral equipment for data acquisition and processing.

An analog-to-digital (A/D) conversion board was installed to handle the data input from the signal conditioning system. This board features a micro-processor for interfacing with the host computer, up to 16 channels of A/D data input, two digital-to-analog output channels, 16 lines of digital I/O, and an on-board programmable clock. A block diagram of the analog-to-digital conversion board is shown in Figure 4-7. This board offers the following operating modes:

- Single Operation - (single event, must be triggered).
- Block Commands - (must be specified, clocked).
- Standard Block Operation. User specifies the number of conversions required, the data channels to be used, and the input gain setting. This is a slow execution mode using BASIC language.
- Continuous Block Operation. As for the Standard Block Operation except that conversions will continue until halted by a "stop" command.
- Block Operation and Direct Memory Access (DMA). Identical to Standard Block Operation except that the data is transferred to and from the host computers memory without intervention. The advantage is a higher data throughput using DMA.
- Continuous Block Operation with DMA. Identical to Continuous Block Operation but with DMA for higher D/A and A/D throughput.

The above modes offer considerable flexibility in data acquisition and any of the modes could prove useful for a specific sensor configuration; however, for the initial testing it was decided to use the Continuous Block Operation mode. In this mode data acquisition is begun when the push is started and is ended when the push is stopped. The data acquisition cycle written for this phase of testing is timed as shown in Figure 4-8. A 1-sec time lapse is equivalent to about 0.8 in. (2 cm) of penetrometer advance, so the limit of resolution of the instrumentation is about 1 in. (2.54 cm). Upon a "start" command this data acquisition cycle is repeated continuously until a "stop" command is given to the host computer. A customized software program was also written to convert the data to engineering units and to plot the output on the host computer CRT display in virtual real time. This is a very desirable capability since the penetrometer operator is able to visually monitor subsurface conditions as the probe is advanced. After the push is ended the data is hard copied while the penetrometer is withdrawn, so there is no need to halt operations for this purpose. However, the chief advantage of real time data acquisition and processing is that results are available immediately and can be used to optimize field operations. For example, a contaminant plume, once detected, could be traced and delineated while the penetrometer is on site.

In general, A/D data converters, such as the one described previously, require input signal levels on the order of ~5 v to achieve adequate resolution. Most sensors cannot deliver such high signal levels without signal amplification,

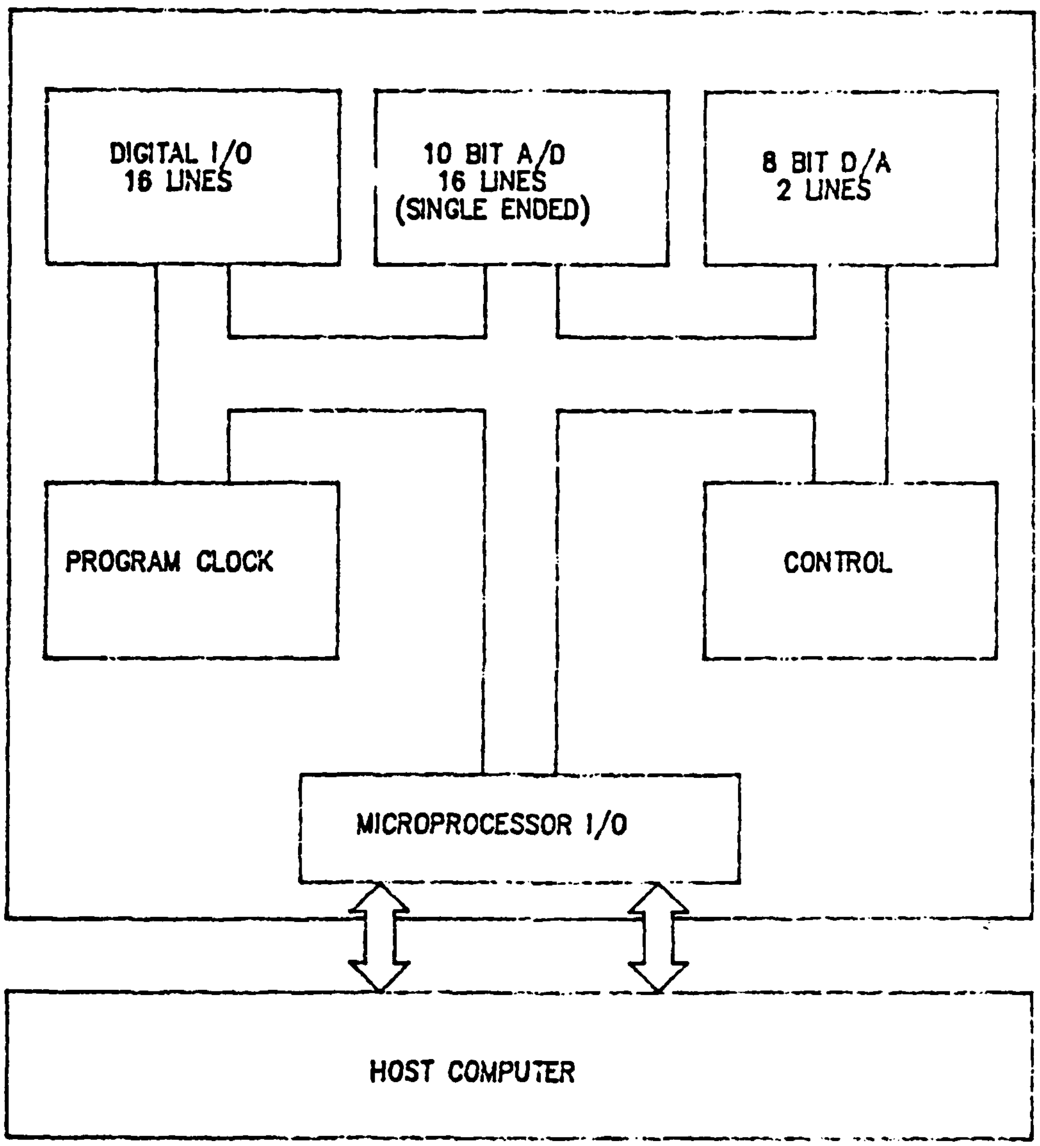


Figure 4-7. Block diagram of analog-to-digital conversion board.

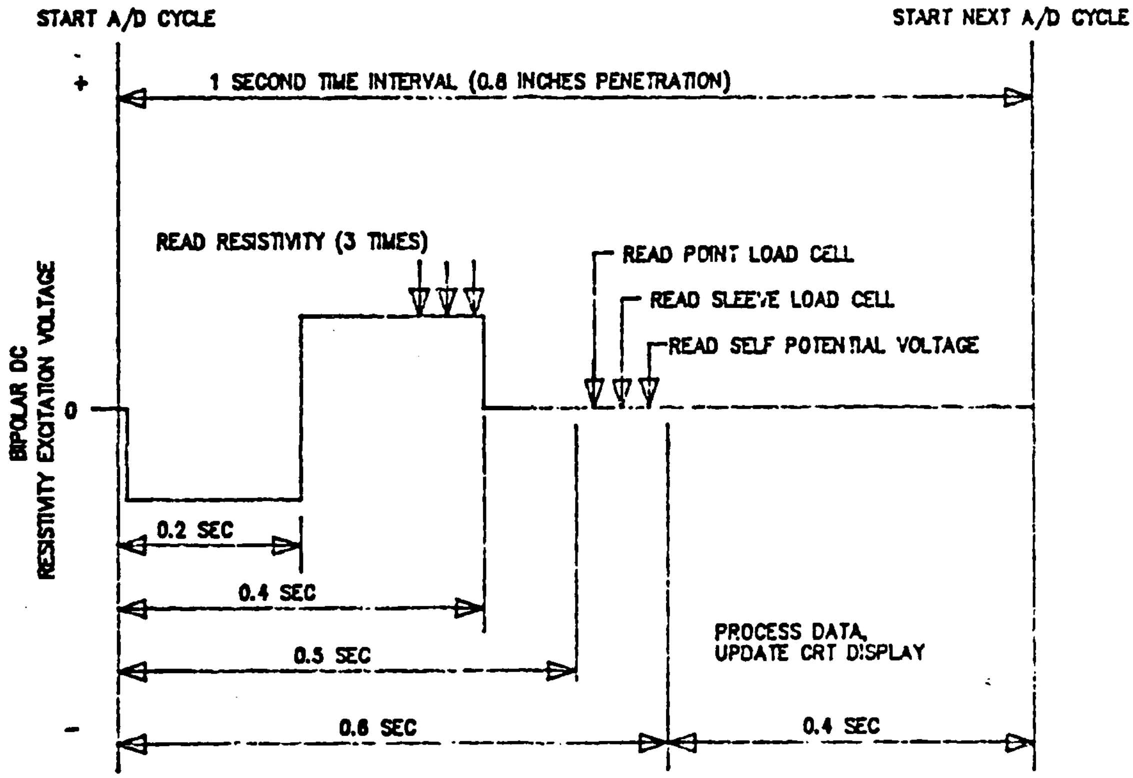


Figure 4-8. Typical data acquisition cycle used in continuous operation mode.

such as would be obtained from instrumentation grade amplifiers with selectable gain settings. Also, when a variety of different sensors are used there is need to have individual signal conditioning for each data channel as well as isolation among channels. For these reasons a signal conditioning unit was developed specifically for multi-sensor cone penetrometer applications.

This prototype multi-sensor cone penetrometer requires four data channels including point penetration resistance, sleeve friction resistance, soil resistivity, and self potential measurements (Figure 4-9). The point and sleeve resistance measurements are made using strain-gaged full bridge load cells (two) which require excitation by a stable 5-v power supply. Output from the bridges was fed to an instrumentation amplifier to achieve the desired signal level for input to the A/D converter. Similarly, a separate power supply was provided to excite the resistivity current electrodes, and the voltage at the resistivity measurement electrodes was fed to an instrumentation amplifier for input to the A/D converter. Although the prototype cone described in this report requires four data channels, provision was made for up to six data channels in the signal conditioning unit. This will allow for additional sensors which may be added in future development work.

It should also be noted that the instrumentation amplifier shown for the self potential measurement circuit did not arrive in time for the proof of concept field tests described later in this report. Consequently, no self-potential measurements were made at that time.

Software for the host computer was written at WES in Microsoft Interpretive BASIC. This format allows for real time editing and debugging which can prevent erroneous data from entering the data files. However, program execution would be too slow using this language so the program is compiled in machine language to achieve about five times faster execution in the computer. This approach has provided the speed needed in data processing to date.

The program as written provides for both data acquisition and processing. The data once obtained are automatically corrected to engineering units. The point and sleeve friction resistance values are used to predict a probable soil classification profile from a soil characterization model (after Douglas and Olsen, 1981) which is shown in Figure 4-10. The point resistance, sleeve friction resistance, soil resistivity, and classification data are all processed in virtual real time and are continuously fed to the CRT for display. Considering the amount of data input to the CRT for display it was necessary to devise a presentation which could provide a meaningful summary of events at a glance. This would permit the equipment operator to continuously monitor all conditions of interest, in particular the instantaneous value of point penetration resistance which would first indicate that the tool had encountered an impenetrable layer (refusal). In this case, failure to stop the push immediately could result in damage to one or more components of the hardware.

It was decided to plot each measurement parameter versus depth, using a series of plotting windows scaled in the appropriate units. In this way all of the data trends, as well as the instantaneous data values, can be seen as the data is updated on the monitor screen. For convenience the data inputs are all

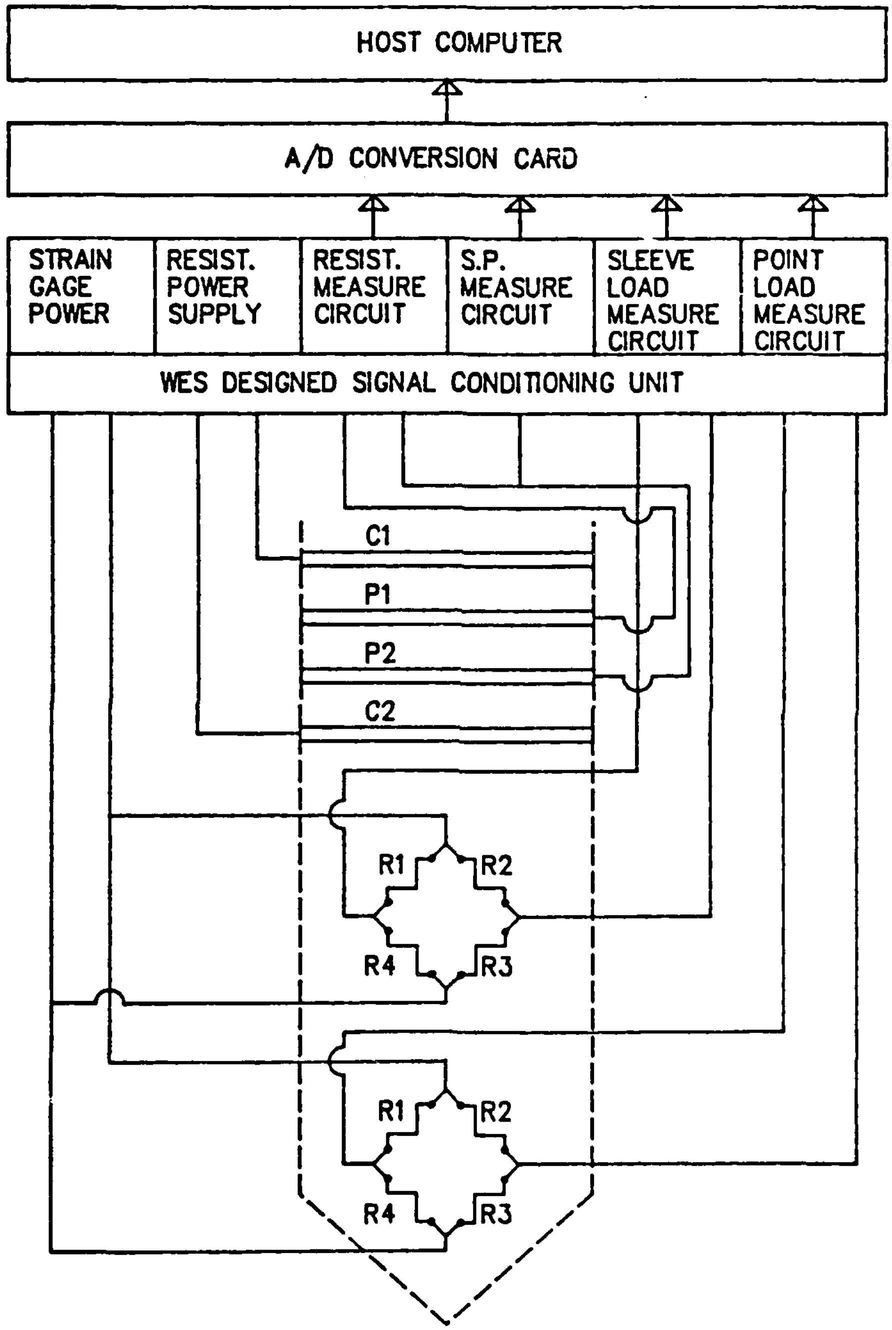
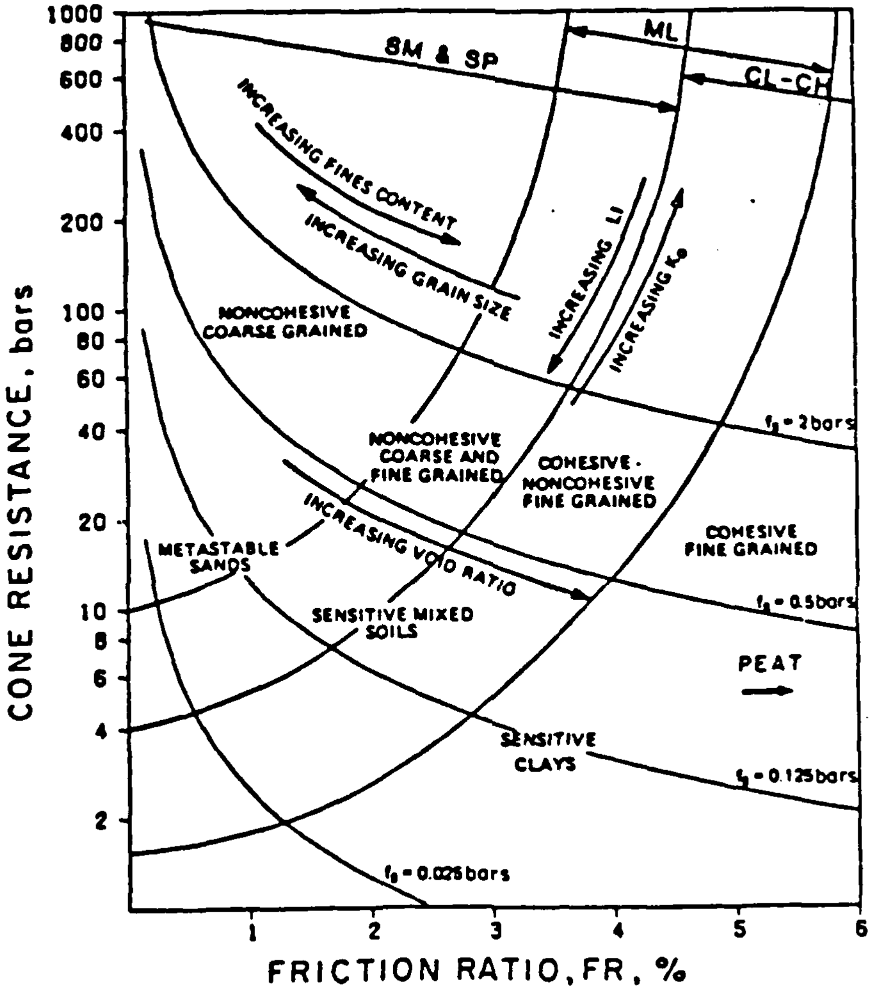


Figure 4-9. Block diagram of signal conditioning circuitry.



1 bar = 100 kPa ≈ 1 kg/cm²

(Adapted from Douglas and Olsen, 1981)

Figure 4-10. Relation of soil properties and soil behavior in testing.

corrected for depth before display, i.e., the geometry of the penetrometer dictates that the point resistance, sleeve friction resistance, soil resistivity, and self potential measurements be made at various distances from the cone tip (hence at different depths during the data acquisition cycle). All of the depth corrections are included in the graphics program.

4.4 Calibration Procedures

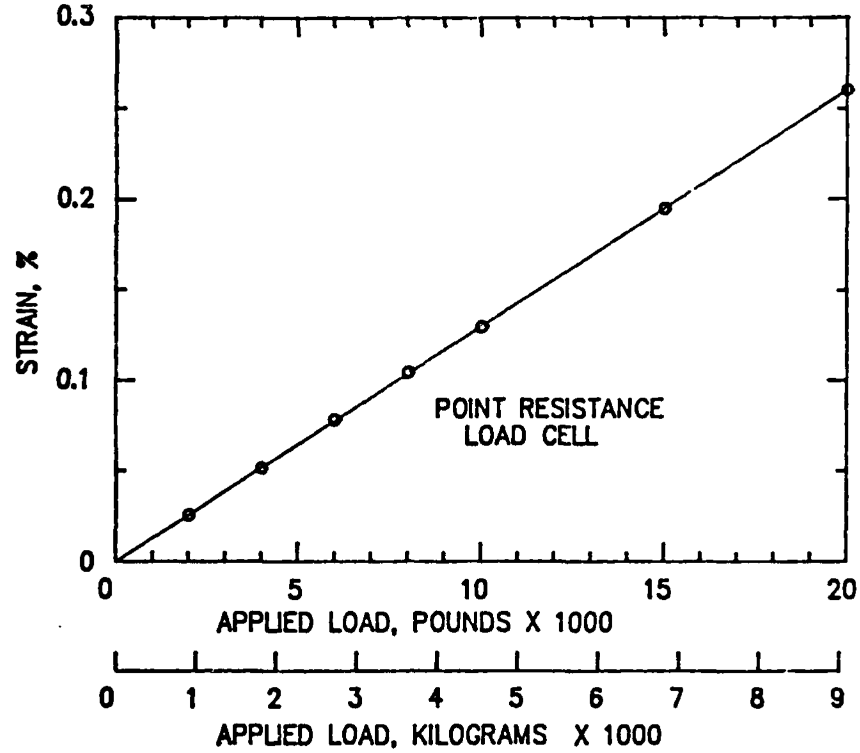
Two calibration procedures are required for the grouting cone load cells. The point penetration resistance load cell is calibrated initially by exercising it several times to its maximum rated compression load of 16,000 lb (7272 kgf). Then, the point load cell is cycled from zero load to a selected load increment and returned to zero load. The load cell output is read for each load increment applied and at the zero load condition at the beginning and end of each loading increment. The load increments are increased until the compressive force reaches the maximum rated capacity of the cell. The friction sleeve load cell is initially calibrated in the same way, except it is loaded to 4,000 lb (1818 kgf) maximum compressive load. Although each load cell is loaded independently, the output of both cells is monitored during loading to detect any undesirable crosstalk. Crosstalk was unmeasureable in the load testing performed to date. Typical calibration test response for the point and sleeve friction load cells is linear to within ~0.5 percent of the applied load, as shown in Figure 4-11. A complete load test is carried out on each grouting cone prior to a field deployment.

Determination of the geometric constant for the resistivity measurement module requires a different calibration procedure. For this calibration the grouting cone, resistivity module, and two lengths of push pipe are assembled as for a field operation. The resistivity module is energized after immersing it in a large reservoir filled with fresh water. With an initial assumption that the geometric constant equals the electrode spacing, the apparent resistivity of the water is measured with the resistivity module. The resistivity of the water is also determined with high precision using a commercial resistivity measuring unit whose accuracy is traceable to the National Bureau of Standards. Next, a premixed salt solution is added, the water in the reservoir is agitated, and a second set of resistivity measurements is made. This procedure is repeated until the response of the resistivity module has been defined by at least five sets of measurements in the domain from 20 to 2,000 ohm-ft (6 to 600 ohm-m). The apparent resistivity read by the resistivity module is adjusted to match the resistivity read by the reference unit using a compensation factor to adjust the geometric constant in the software program. A geometric constant of 1.32 times the electrode spacing is appropriate for the prototype resistivity measurement module, as shown in Figure 4-12.

5.0 FIELD TESTING

5.1 Site Description

Proof of concept field testing of the cone penetrometer system was carried out at Louisiana Army Ammunition Plant (LAAP), near Minden, AL. This facility is within 150 miles (250 km) of WES, and offered the advantages of convenience and accessibility. The facility is operated by Morton Thiokol, Inc., whose



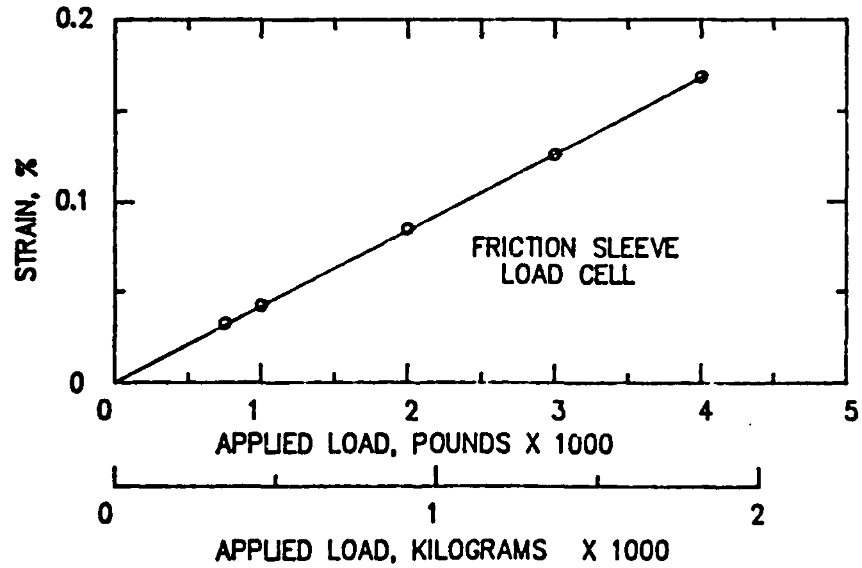


Figure 4-11. Typical calibration test results on grouting cone load cells.

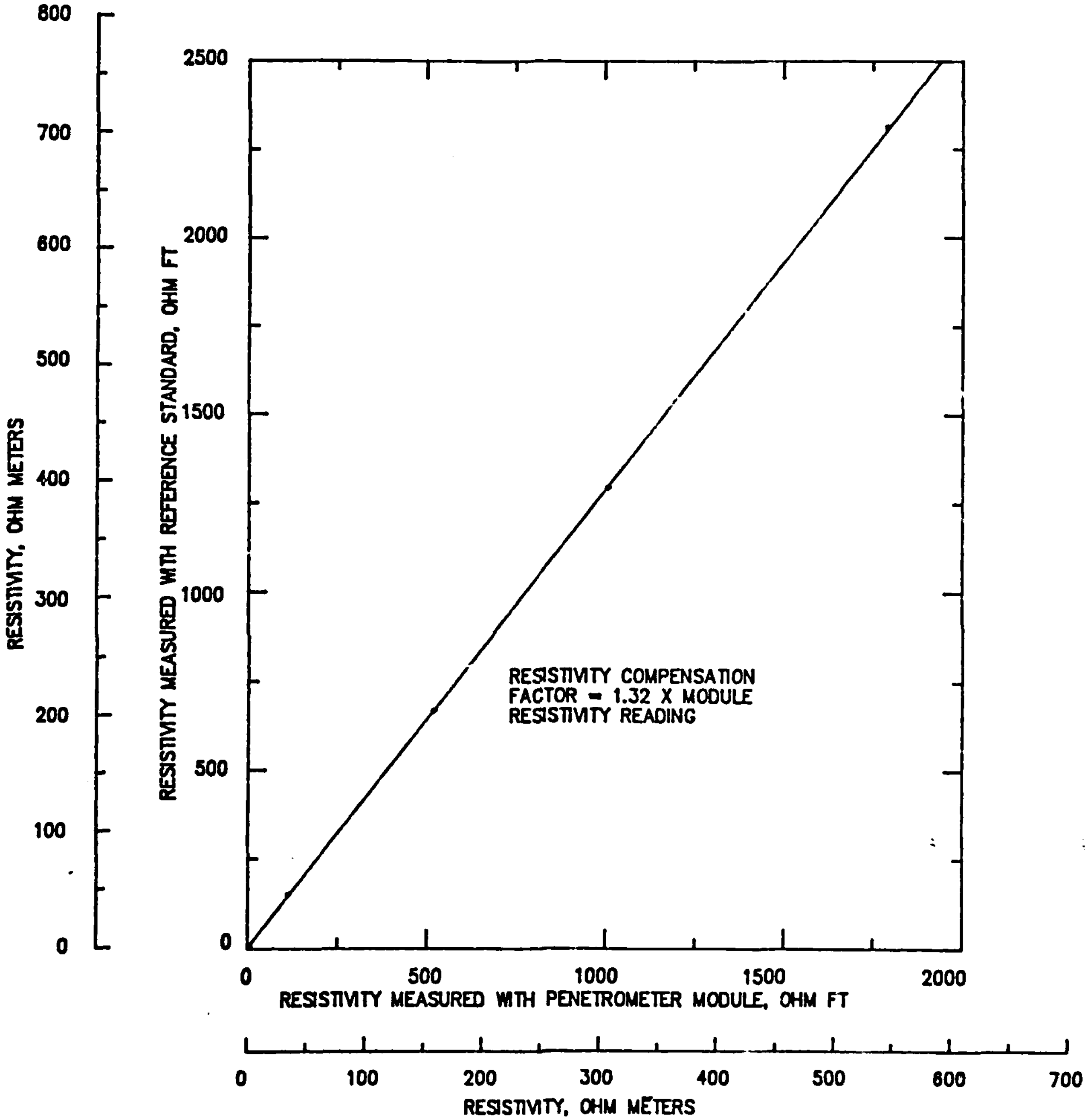


Figure 4-12. Typical calibration test response for resistivity module.

Supervisor for Environmental Protection, Mr. David Burroughs, coordinated the field tests. Mr. Burroughs recommended the southern part of Inert Disposal Area No. 3 as a test site, and his suggestion was adopted.

The test site area is forested with pine trees and is characterized by gently rolling topography with less than 10 ft of relief in the test area. Figures 5-1 and 5-2 show the location of Inert Disposal Area No. 3. The disposal area was used as a evaporation pond for neutralized alkaline wastes in World War II and was later used as a burial ground for inert waste. The site was filled and closed but leveling of the area for monitoring well installation uncovered some inert explosives waste (brownish-black, and caked) and badly deteriorated drums. In 1982, an evaluation of the hydrogeologic conditions at the site indicated that the area did contain residue related to the evaporation pond, but records available on ground water composition did not indicate any pollutants because the monitoring wells were not down the ground-water gradient from the active part of the site (Envirodyne Engineers, Inc., 1982).

The water table as measured in 1982 was approximately 178 ft (58.7 m) above mean sea level (MSL) in the area of the traverse. The ground surface elevation near monitoring well G56, located 91.5 ft (30.2 m) southwest of Penetrometer Hole No. 7, is 211.2 ft (69.7 m) above MSL. The water table was only 33.2 ft (10.9 m) below ground surface in 1982. the ground water flow direction was determined to be to the northeast with a gradient of 0.00248. The maximum horizontal flow velocity was estimated at 7.5 ft/year (2.47 m/year). Based on the direction and flow velocity, the pollution should have moved approximately 350 ft (107 m) during the 40 years since waste was placed in the area. The traverse covered in the present study crosses an area that overlies any pollution plume extending northeastward from the old evaporation pond and landfill. Penetrometer Holes 1 and 2 should be out of the immediate pollutant plume but Holes 3, 4, 5, 6, 7, and 8 are in areas strongly suspected of being infiltrated by waste water or leachate from solid wastes.

New monitoring wells were installed in April 1986 and the driller's logs show ground water was encountered at a depth of 15 ft (4.9 m) at monitoring well G0122. G0122 is approximately 275 ft (90.22 m) northeast of the line of traverse. The driller's log for G0122 is presented in Table 5-1. It is instructive to note that 1.5 ft (0.46 m) samples were taken at 5 ft (1.52 m) intervals in boring G0122. This kind of skip sampling is standard practice when conducting exploratory borings, and serves to illustrate the tendency to minimize the number of samples taken as one measure to reduce costs of conventional drilling, sampling, and laboratory sample testing programs. However, this kind of sampling cannot provide a meaningful index of subsurface conditions when soil layers are thin, interbedded, and discontinuous. If a detailed and continuous record of soil conditions is not obtained then monitoring wells may not be correctly placed, as was the case for the original monitoring wells at this site. Continuous sampling is needed to more accurately define the soil profile, but is not usually attempted because of the expense involved, particularly in an extensive drilling program.

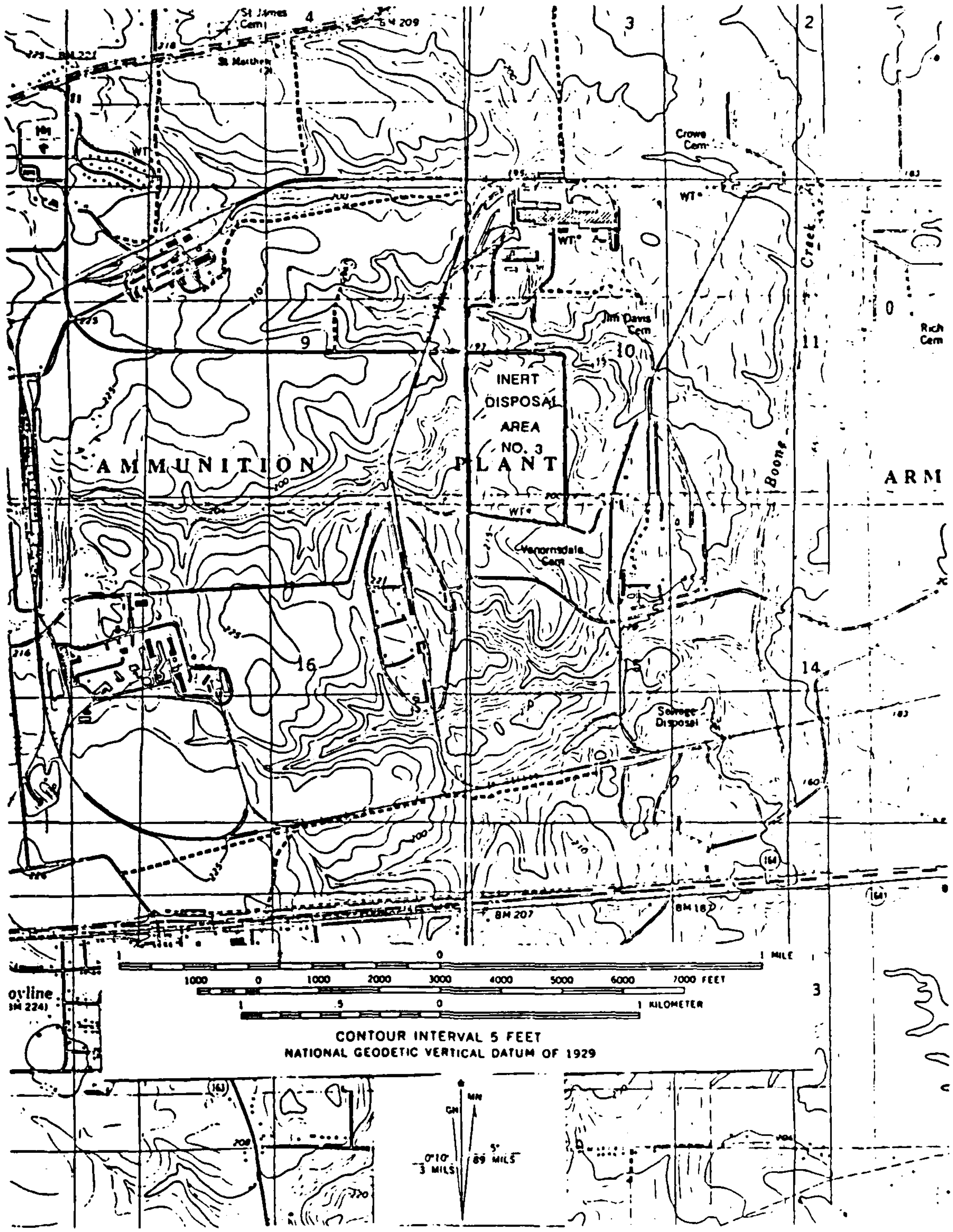


Figure 5-1. Location of Inert Disposal Area No. 3.

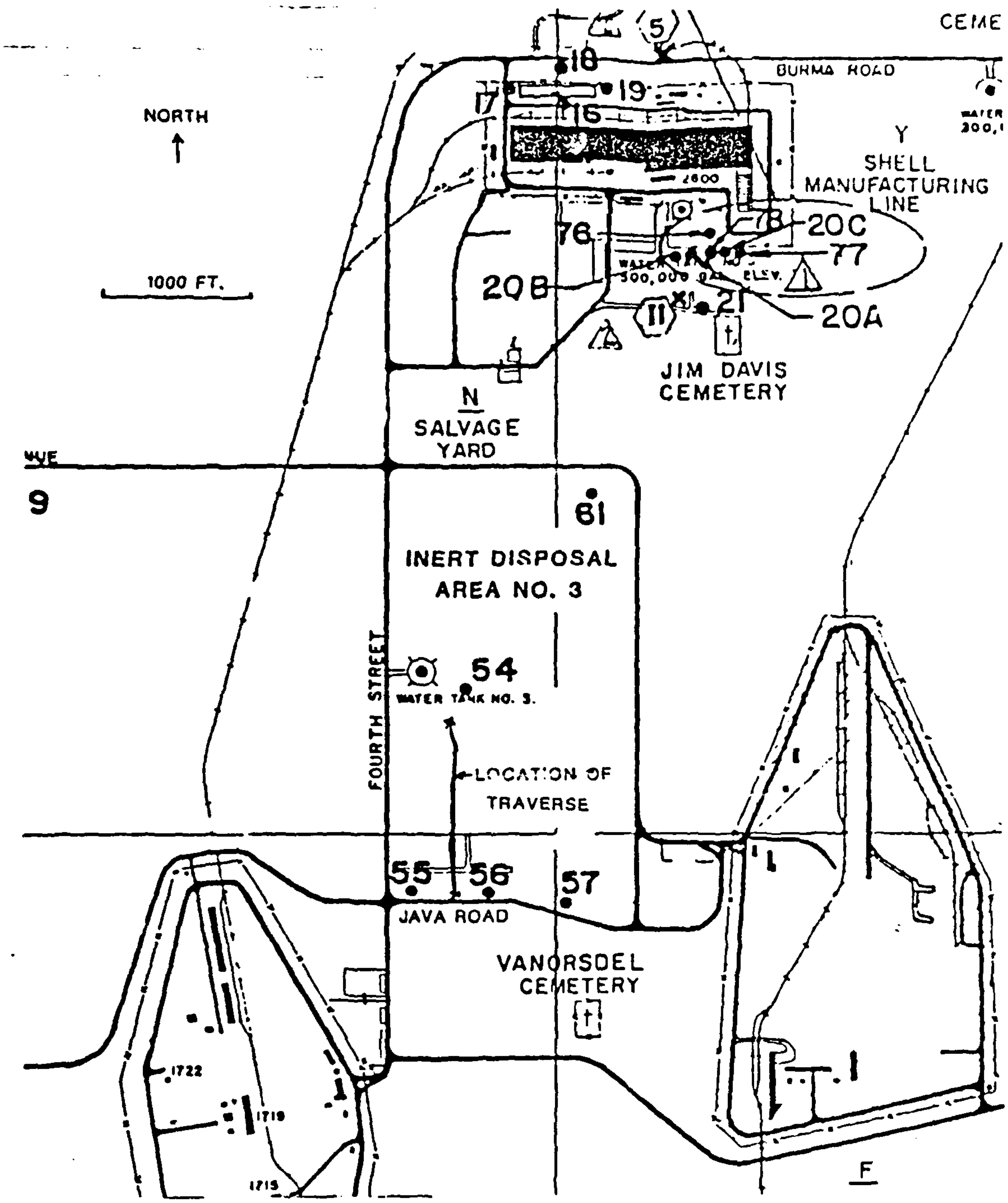


Figure 5-2. Location of traverse in Inert Disposal Area No. 3.

Table 5-1

Driller's Log for Monitoring Well G0122

Depth, ft (meters)	Lithology
5-6.5 (1.64-2.13)	Silty clay 25 percent silt 5Y6/2 (light olive gray) to 7.5YR5/8 (strong brown), mottled, moderately plastic, stiff damp
10-11.5 (3.28-3.77)	Clay, (2.5YR3/6) dark red, very plastic, stiff, damp < 5 percent organic smears
15-16.5 (4.57-5.05)	Silt, w/very fine quartz sand, 30 percent silt, (5YR5/8) yellow red, non-plastic, dense wet (first free water encountered)
20-21.5 (6.09-6.55)	Silt (as above) interbedded with clay (see 10-11.5 ft)
25-26.5 (7.62-8.08)	Sand, very fine to fine, 7.5YR7/8 (red-yellow) non-plastic, dense, poorly-graded, damp
30-31.5 (9.14-9.60)	Clay, slightly silty, 10 percent silt, 10 percent organic matter (well-distributed smears and modules (5Y7/2) light gray to (7.5YR5/8) strong brown, mottled, very plastic, stiff, damp
35-36.5 (10.67-11.12)	Clay, as above with less organic matter (5 percent)
40-41.5 (12.19-12.65)	Clay, mostly (5YR4/6) yellow-red with some (5Y7/2) lt. gray mottled, very plastic, stiff, damp
45-46.5 (13.71-14.17)	Sand, very fine to fine quartz (10YR7/6) yellow, non-plastic, dense, poorly graded, wet slightly clayey in places
50-51.2 (15.24-15.69)	Sand, slightly gravelly fine to medium quartz sand 5 percent gravel to 2 cm diam (10YR6/6) brown-yellow, non-plastic dense, moderately graded, wet
55-56.5 (16.76-17.22)	Sand, gravelly clayey quartz, 10 percent gravel, 10 percent clay (10YR7/6) yellow, non- to slightly-plastic, dense, wet
60-61.5 (18.28-18.74)	Sand and clay, interbedded with clay and gravel (10YR7/8) yellow gravel with many colors non-plastic to slightly plastic dense
65-66.5 (19-20.27)	Sand, slightly gravelly, very fine to fine quartz, 5-10 percent gravel, (10YR7/6) yellow non-plastic, dense, moderately graded wet

(Continued)

Table 5-1 (Concluded)

Depth, ft (meters)	Lithology
70-71.5 (21.33-21.79)	Sand, fine to coarse, gravelly, clayey quartz, 0-20 percent gravel, 0-20 percent clay, (10YR7/6) yellow, non-plastic to moderately plastic dense, wet
72-73.5 (21.94-27.4)	Clay, silty, 20 percent silt, (5Y2.5/1) black, finely laminated glauconitic in places, very plastic, hard, damp

5.2 Penetrometer Tests

The penetrometer system was calibrated at WES on 9-10 August 1986 and transported to LAAP on 12 August. Eight penetrometer holes were pushed along a traverse that ran from the central portion to the southern edge of the disposal area (Figure 5-3). The penetrometer investigation was begun 16 August and concluded on 22 August. The push rig employed in this phase of the research was a truck-mounted unit with two hydraulic rams and a reaction mass of 15 tons (13.7 metric tons). Figures 5-4 and 5-5 show the unit in the field.

At each push location, the truck with the hydraulic push rig aboard was positioned over the survey mark and leveled using blocks and jacks. The penetrometer was then cleaned and inspected and fitted with a sacrificial tip. The lines for grouting were examined for crimps or blockage (see Figure 5-5). The cable from the equipment van was hooked up and the instrumentation was checked out. The pressure injection system for the grouting was checked and the tanks were refilled to capacity.

The progress of the push was followed in the instrument van using a two-way radio for voice communication. The beginning and end of each 39-in. (1-m) rod segment was called out and variables such as hydraulic cylinder push pressure and any rod flexure were noted. The speed of the rod advance was maintained at approximately 0.07 ft/sec (2 cm/sec).

The penetrometer push was halted if the 15 ton (13.7 metric ton) WES push rig reached its maximum capacity (refusal) and lifted up off of the support jacks. The cone was then raised a few inches from the bottom of the hole, the grouting was initiated and the diluted urethane pre-polymer solution and the water were injected into the hole as the grouting cone was raised. The grout pressure in the hole was maintained at 20 to 30 psi (138-207 kPa) as the cone was withdrawn. The grout foams as it gels and the action of the grout could be observed as the solid foam column pushed to the surface after the penetrometer was withdrawn. Approximately three gal (12 ℓ) of grout was required to seal each hole. The grout take on each hole was approximately 10 percent greater than the volume calculated to fill the nominal hole void. In general, this indicated the geologic material surrounding the hole was relatively impervious and that the grout was gelling very rapidly. Circumstances did not permit an excavation at the LAAP site, but the grout take at that site is evidence that these penetrometer holes were also well sealed.

After grouting was completed the grout lines were flushed with acetone. The acetone was recycled to a storage vessel for use in the next flushing operation. The grouting cone was then cleaned of any adhering grout and the push rig was moved to the next surveyed point. The availability of real-time data from the penetrometer permitted immediate comparisons among penetrometer pushes as the traverse progressed.

5.3 Conventional Soils Property Testing

Continuous split spoon sampling was done in boreholes drilled 3 ft (0.9 m) from each penetrometer hole at four of the seven penetrometer push sites. Borehole 1 was located between the two duplicate penetrometer pushes (1 and 2).

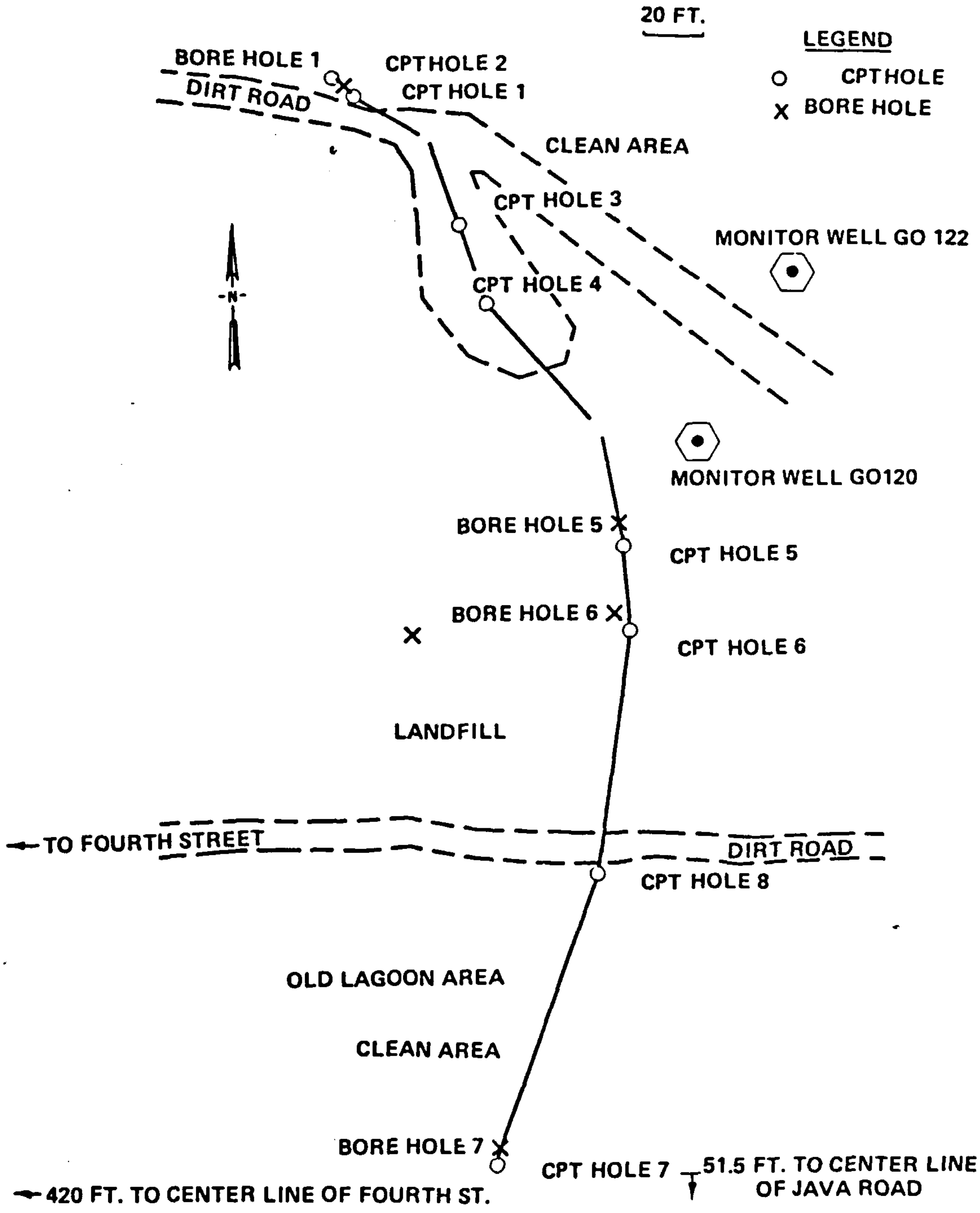


Figure 5-3. Location of boreholes and penetrometer (CPT) holes along the traverse line.

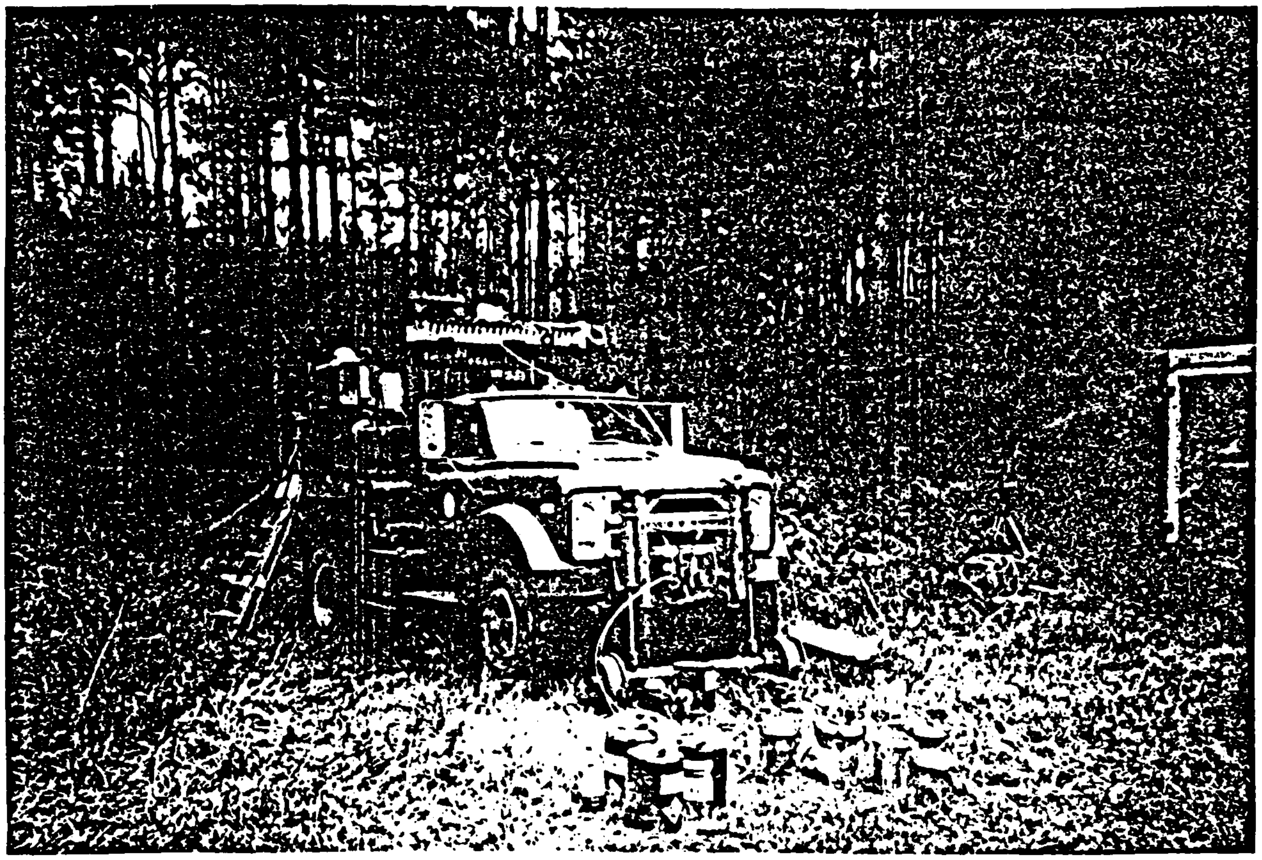


Figure 5-4. Hydraulic push rig on site at LAAP.

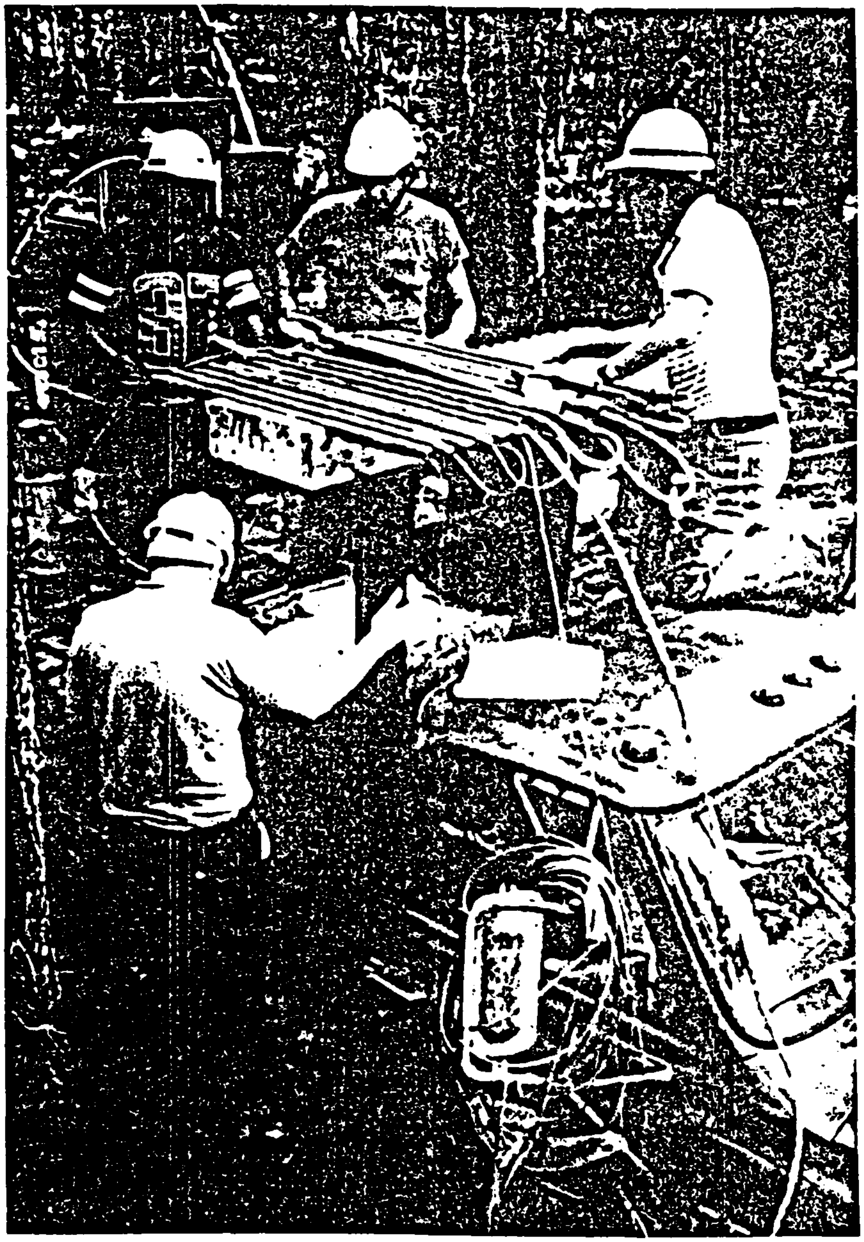


Figure 5-5. Crew rigging instrument cables and grouting lines prior to initiating a push sequence.

At each sample point a penetration test was made using the conventional 140-lb (63.5-kg) hammer dropped from 30 in. (76.2 cm) and a 2-in. (5-cm)-diam sampler. The penetration testing was done as nearly continuously as possible in order that none of the vertical interval investigated with the probe went unsampled. The blow count was tabulated for each full 18-in. (45.2-cm) interval, and no portions of the sampled soil were discarded or eliminated from the sample. Each 18-in. (0.6-m) interval sample was removed from the split-spoon sampler, examined visually, described, and packed in air-tight containers.

The samples contained waste materials and, until the mild alkalinity of the soil was established, handling of the samples was kept to a minimum. The soil samples from boreholes 1, 5, and 6 were examined at the laboratory and were halved to provide duplicate subsamples. One set of subsamples was subjected to standard soil tests for classification including determination of Atterberg limits and a sieve analysis. The samples were classified based on the Unified Soil Classification System (USCS). The methods employed are described in Appendix A.

Conventional penetration test procedures would have required that only the blow count on the lower 12 in. (30.48 cm) be used to represent the entire interval. Conventional sampling procedures also would have required that 1 ft (30.48 cm) of soil be removed between each split-spoon drive (Water and Power Resources Service, 1974). Drilling out 1 ft (30.48 cm) would have resulted in a loss of approximately 30 percent of the soil data. The modified procedure resulted in all of the soil column being tested with no skipped intervals and all recoverable soil being included in the samples to be tested.

The second subset of samples was subjected to laboratory analysis to determine the slurry pH and the soil conductivity. The methods used are described in Appendix B.

6.0 COMPARISON OF PENETROMETER AND CONVENTIONAL TEST RESULTS

6.1 General

The intent of the limited drilling, sampling, and laboratory sample testing program in this study was to provide physical evidence of subsurface conditions for comparison with the penetrometer results. Time and funding constraints precluded an intensive verification program such as continuous undisturbed sampling at each cone penetration location, strength and material property testing of samples from each layer encountered, and a full suite of chemical analysis testing. A cost effective compromise was to conduct continuous drive sampling with a split spoon sampler at four cone penetration location, i.e., two inside the contaminated area and two outside. This approach provided blowcount information for comparison with the cone penetration test strength results, and also provided disturbed samples for laboratory determinations of some soil properties of interest, i.e., soil classification, pH, and resistivity. Three of the four boreholes were selected for characterization. Samples covering the upper 25 ft (7.6 m) of boreholes 1, 5, and 6 were studied and compared to the data collected from the penetrometer in the same depth intervals.

It is important to consider the way in which the various test procedures can influence the comparisons to follow. For example, the accuracy of depth interval determinations in the various tests used are approximately as given in Table 6-1.

As a practical example of the influence of depth and depth interval resolution, consider the hypothetical soil profile shown in Figure 6-1. In this example, two 6-in. (15.2-cm)-thick sand lenses occur in a thick clay section. Note that in no case does a routine laboratory classification test on an 18-in. (45.7-cm) split spoon (disturbed) sample produce results which reflect the actual profile and soil type in the hypothetical depth interval containing the sand lenses. Strict adherence to soil sampling procedures would require a separate sample for each discrete layer, when such layers can be distinguished visually. The requirement in this investigation for splitting samples for chemical testing and soils classification made it impractical to collect samples over intervals less than 18 in. (45.7 cm) long. Shorter intervals did not provide enough sample to assure reliable testing.

Figure 6-1 shows how the requirement for an 18-in. (45.1-cm) sampling interval can affect the classification of the soil when samples from thin-layered, interbedded or discontinuous soil media are present at the site. The penetrometer results can show a detailed view of thinner (6 in. or 15.2 cm) layers within the larger sampled interval. The description of the overall interval can be altered by the position of the actual sample within layers in the interval.

Another factor to be considered is the comparison of field and laboratory resistivity results. The cone resistivity module measures the bulk resistivity of the surrounding soil, insitu, and these measurements are accurate to about 2 in. (5 cm) in depth. The standard laboratory soil resistivity measurement is made on an 18-in. (45.7-cm)-length disturbed soil sample mixed with distilled water to a pasty consistency (see Appendix B). Distilled water has a very high resistivity because any salts, mineralization and impurities have been removed. Adding distilled water to produce a paste-like consistency of the laboratory soil sample obviously does not replicate the in situ (denser) condition, and also tends to significantly increase the water content of a given volume of sample. Hence, the laboratory resistivity measurement is biased towards a higher resistivity value than exists insitu. It follows that the comparison of resistivity results from the insitu and laboratory tests should be made on the basis of trends, as a function of depth, rather than comparing absolute values (the laboratory test will typically indicate a significantly higher value of bulk soil resistivity).

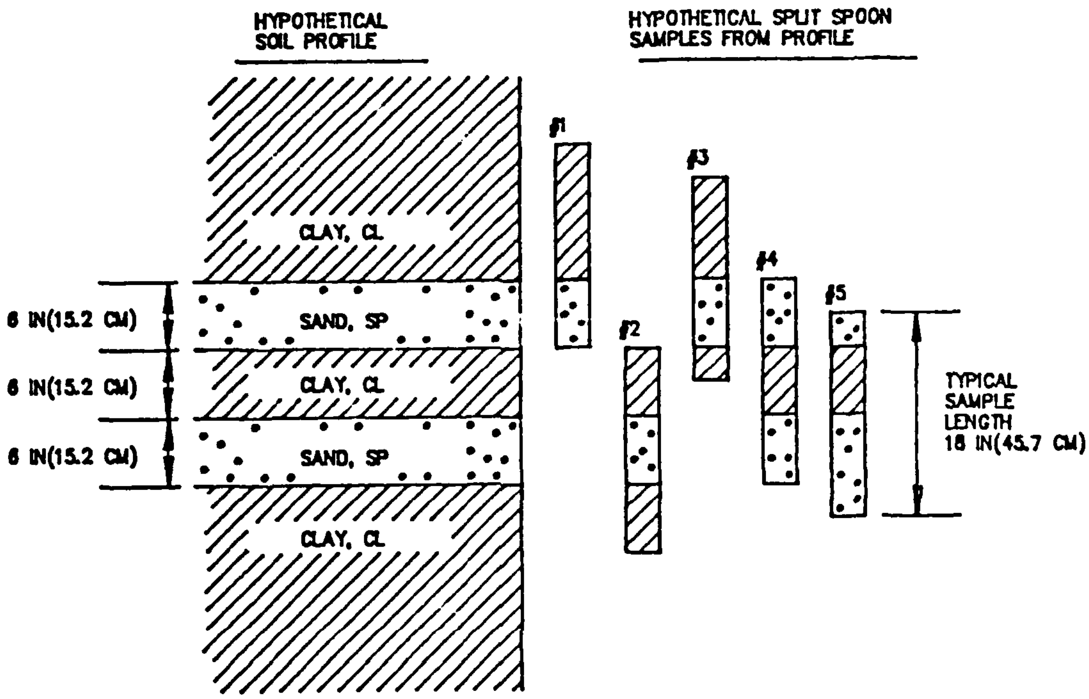
6.2 Strength Properties

The cone penetrometer develops strength data and a soil characterization from the cone pressure and sleeve friction. Figures 6-2, 6-3, 6-4, 6-5 present a comparison of the penetration test blow count, an index of soil strength with penetrometer data for penetration to a 25-ft (7.6-m) depth in boreholes 1, 5, 6 and 7. Each borehole is approximately 3 ft (0.9 m) from the probe hole with the same number (Figure 5-3). The separation is great enough at this shallow penetration that the sealed penetrometer hole should have no significant effect on the borehole penetration test results.

Table 6-1

Test Methods and Practical Limits of Resolution for Soil Parameters

Test Method	Measurement Parameter	Practical Limit of Resolution
Cone	Depth to boundary between soil layers (stratigraphy)	Resolves to within 0.8 in. (2 cm), using point penetration resis tance measurement. The cone point measurement of resistance to penetration is influenced by layers above and below (to perhaps 3 to 5 cone diameters) but the crossing of a layer boundary should still be detectable.
Cone	Soil Classification (soil type)	Friction sleeve is 5.2 in. (13.3 cm) in length, so soil layer must be of greater thickness, perhaps 6 to 8 in., to classify soil type using point and sleeve resistance values.
Cone	Penetration Resistance (soil strength)	Resolves to within about 6 in. (15.2 cm) using point penetration resistance measurement.
Cone (Resistivity Module)	Bulk Resistivity of Soil	Resolves to the electrode spacing of 2 in. (5 cm).
Laboratory Resistivity	Bulk Resistivity of Disturbed (reconstituted) Soil Samples	12 in. (45.7 cm)
Standard Penetration Test (SPT)	Blow Count, N	18 in. (45.7 cm)
Split Spoon Sampling	Soil Properties from Sample	18 in. (45.7 cm) (Sample Testing)



LABORATORY CLASSIFICATION TEST RESULTS FOR FULL LENGTH HYPOTHETICAL SAMPLES

SAMPLE NO.	SOIL TYPE BY VOLUME	PROBABLE CLASSIFICATION
#1	67% CL, 33% SP	SANDY CLAY, CL
#2	67% CL, 33% SP	SANDY CLAY, CL
#3	67% CL, 33% SP	SANDY CLAY, CL
#4	67% SP, 33% CL	CLAYEY SAND, SC
#5	50% CL, 50% SP	SANDY CLAY, CL OR CLAYEY SAND, SC (?)

Figure 6-1. Hypothetical soil profile demonstrating the problem of classification of thin soil layers.

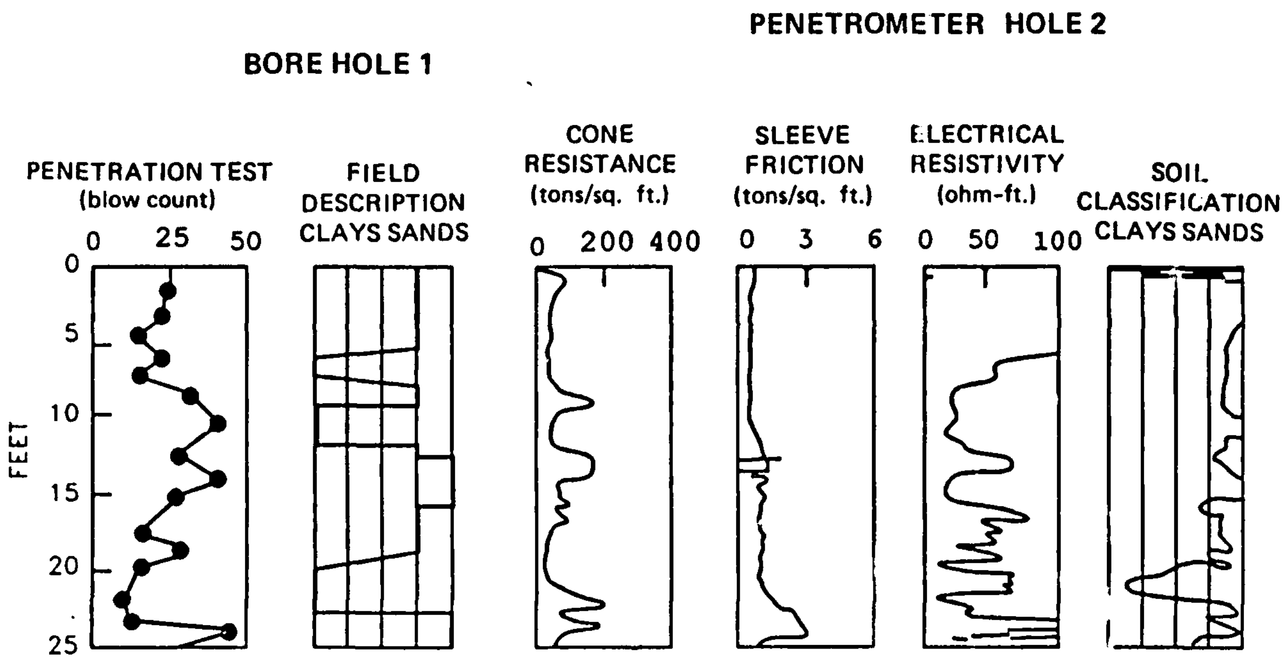


Figure 6-2. Comparison of data from borehole 1 and data from cone penetrometer hole 2.

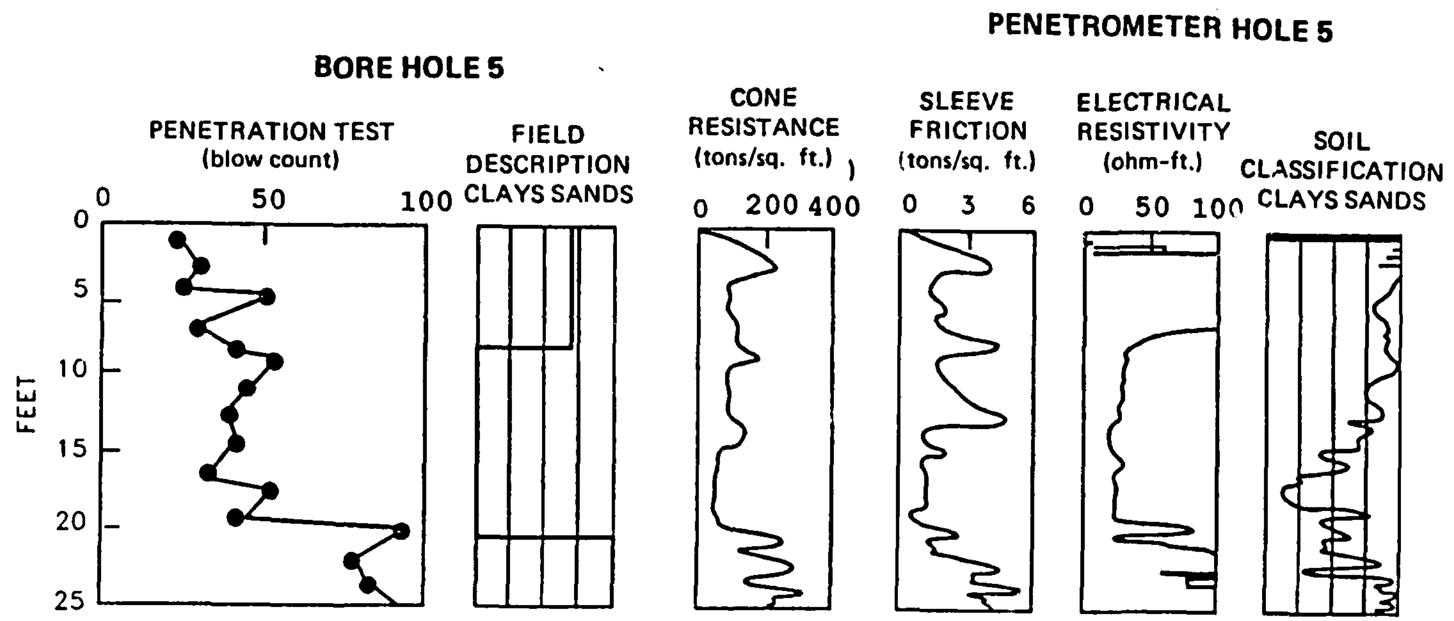


Figure 6-3. Comparison of data from borehole 5 and data from cone penetrometer hole 5.

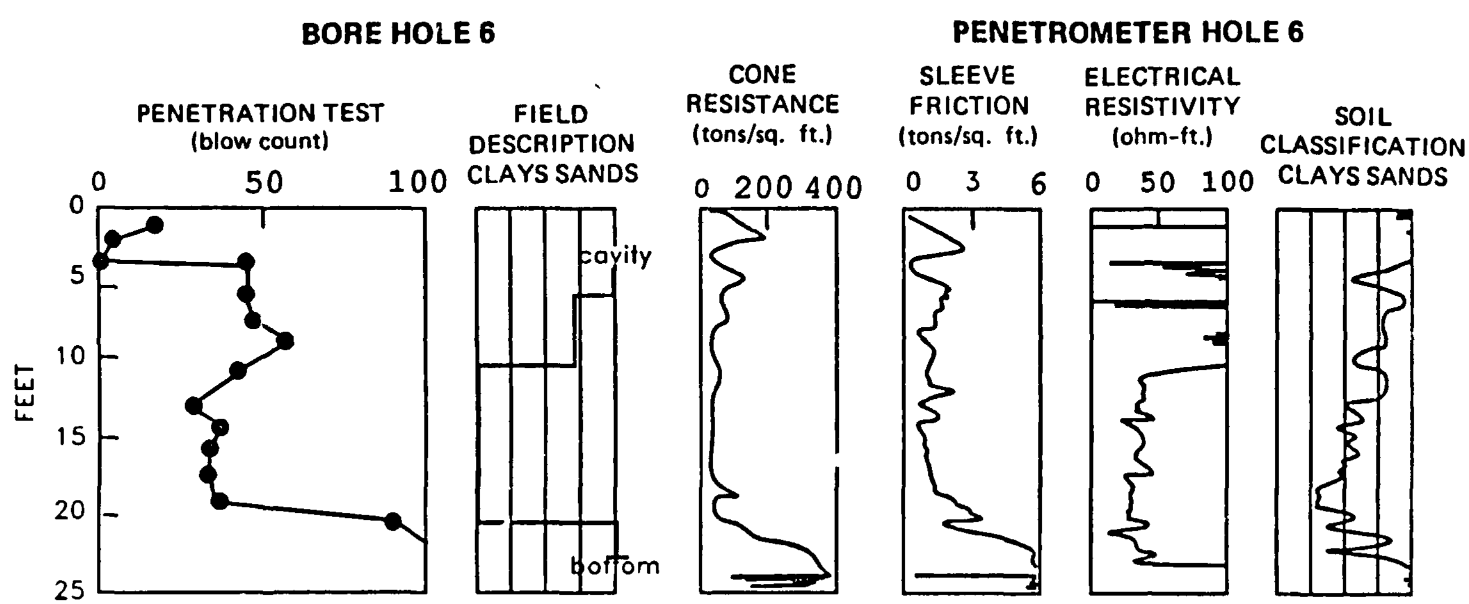


Figure 6-4. Comparison cf data from borehole 6 and data from cone penetrometer hole 6.

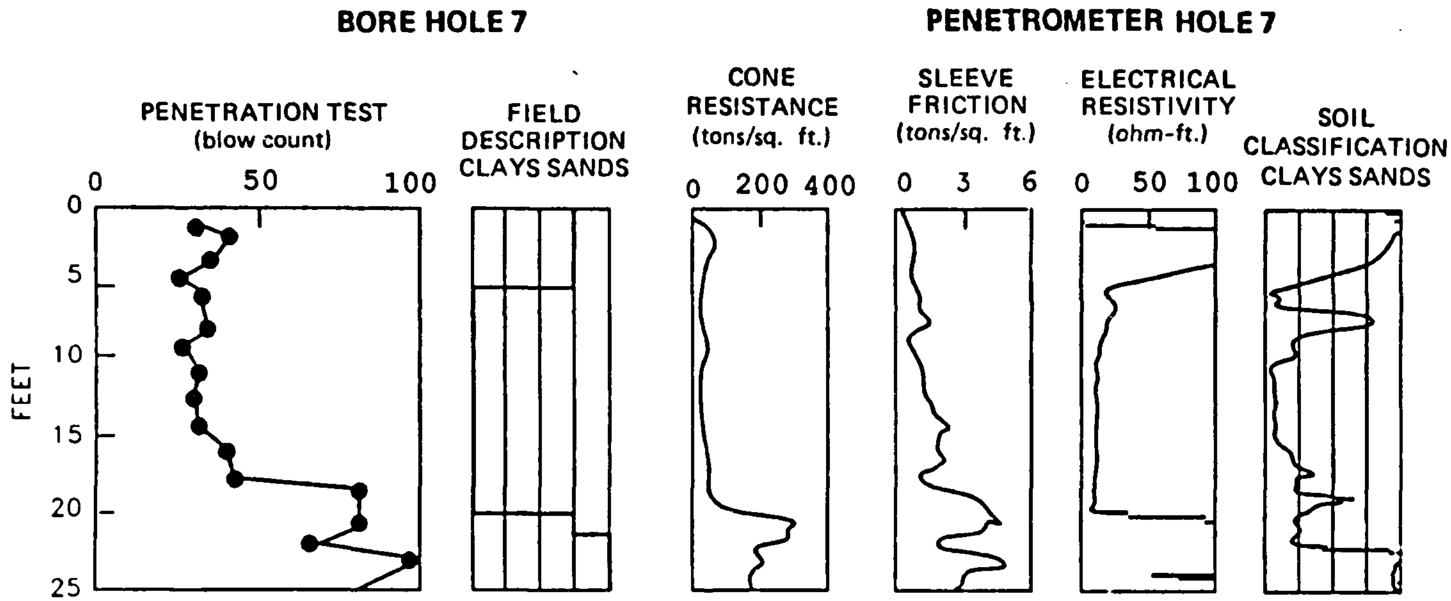


Figure 6-5. Comparison of data from borehole 7 and data from cone penetrometer hole 7.

In borehole 1, the penetration test (blow counts) reflects the trends noted in the cone penetration resistance. The penetration test variation is over a coarse interval of 18-in. (45.7-cm) length, while the cone penetration resistance records change to within about 0.8 in. (2 cm). The prominent high-strength layers at approximately 10.5 ft (3.2 m), 13.5 ft (4.1 m), and 24.0 ft (7.3 m) are well-documented in both the penetration test and cone resistance readings.

In borehole 5, the penetration test record shows only one major increase in blow count at approximately 6-ft (1.8-m) depth. The cone shows this feature and gives more detail in the upper part of the section. The extremely strong layer at the 21-ft (6.4-m) depth is clearly evident in both the penetration test and cone resistance data and corresponds to a major change in soil strength.

Borehole 6 shows a useful correspondence of penetration test and cone resistance data. The cavity noted at the 3-ft (0.9-m) depth gives a zero blow count and the cone penetration resistance drops to near zero. The penetration test shows refusal at 21 ft (6.4 m). The penetrometer similarly went off-scale (over 400 tsf or 390 kgf/cm^2).

In borehole 7, the most prominent feature is a high-strength layer that appears at the 19-ft (5.8-m) depth. The cone penetration resistance documents the character of the layer in more detail than the penetration test. Both measurement systems show similar trends.

Strength measurements made by penetration test and the cone penetrometer are comparable. All of the major intervals characterized by strength contrasts could be found using either borehole penetration test or the cone. The cone measurements give more detail on strength variations than the borehole penetration test.

6.3 Stratigraphy

The soil stratigraphy (layering) and soil type are important features in any site investigation. The arrangement and thicknesses of different types of subsurface materials controls the pattern of contaminant migration. Stratigraphic considerations become critical in decisions as to where to locate sample borings or monitoring wells and where to set screens in these wells.

In stratigraphic investigations the cone offers several advantages over the drive samplers. The resolution of soil interface using the cone point penetration resistance is on the order of 0.8 in. (2 cm) as opposed to 18 in. (45.7 cm) for the drive sampler. Continuous cone resistance measurements provide much more detail and resolve thinner bedded units missed by the drive sampler. Table 5-1 shows a typical log for a borehole in the test area. The most prominent features in the soil column are the silty clay in the upper 6 ft (1.8 m) and the intermediate clay and silt units from a depth of 10 ft (3.05 m) to a depth of 22 ft (6.7 m). Dense sands appear in the section near 25 ft (7.6 m). Both the penetration test and the cone penetration resistance logs show these major stratigraphic features.

In penetrometer holes 5 and 6, located on the landfill the cone penetration resistance and the penetration test in the upper 4 ft (1.2 m) show a high penetration resistance. The penetrometer holes at 2 and 7 located off the landfill show both lower penetration resistance and sleeve friction. The penetration data indicate the compacted landfill cap extends through penetrometer holes 5 and 6, but is missing at penetrometer holes 2 and 7. Units lower in the soil section such as the dense sand at the 21- to 25-ft (6.4- to 7.2-m) depth could be traced across the entire site.

6.4 Soil Classification

The soil classification presented by the CPT (or cone penetration test) program in Figures 6-2 to 6-4 are generated by a classification algorithm that takes the cone penetration resistance and the sleeve friction and develops a soil classification from curves similar to those shown in Figure 4-8. Tables 6-2 to 6-4 show tabulations of the results of three separate techniques for soil classification; description of samples in the field, laboratory testing for grain-size and plasticity (Atterberg limits) with subsequent classification in the Unified Soil Classification System (USCS), and classification based on cone point resistance and sleeve friction in the CPT.

In the field description, categories are broad and based on visual estimates only. The laboratory-derived classification is developed around a compartmentalized descriptive identification system that depends on specific parameters developed in the sieve analysis and plasticity testing. The CPT classification is developed from the behavior of the soil when an instrumented penetrometer is pushed through the soil in situ. The classification systems operate with different inputs and the resulting classifications will not necessarily agree.

In penetrometer hole 2, (Table 6-2, Figure 6-2) the computer algorithm for classifying samples as to soil type selected a sand designation for soils in the upper 18 ft (5.48 m). The field description and subsequent laboratory test classified the soils as clays. Examination of samples indicate the clays are strongly cemented. The cementation produces unusually high cone penetration resistance and causes anomalous responses from the sleeve as it is forced past rough cemented layers. The existing classification algorithm is not able to produce useful classification designations in such materials. Better performance could be expected in normal soil conditions. Below the 19.5-ft (6.44-m) depth, cementation was not significant and classification from the CPT and laboratory testing agree reasonably well.

In the area of borehole 5 (Table 6-3) the CPT classified the upper 7.5 ft (2.3 m) of the soil as sand, the field classification called the interval silt and the laboratory classification indicated the soil was silty or sandy clay. This upper part of the hole was in the filled waste pond. The sand classification was produced by the high cone penetration resistance, but low sleeve friction from the recompacted fill. Below the fill (12 ft or 3.6 m depth) the classification from the cone agrees with the laboratory classification and the visual or field classification.

Table 6-2

Summary of Laboratory and Field Data from Borehole 1 and Penetrometer Holes 1 and 2

Depth to Top of Interval (ft)	Field Description	Laboratory Classification USCS	Cone Classification	pH	Slurry Resistivity (ohm-ft)	Penetrometer Hole 1 Average Resistivity (ohm-ft)	Penetrometer Hole 2 Average Resistivity (ohm-ft
0.0	silt	CL	sand	5.10	219	0.1	1,280.0
1.5	silt	CL	sand	5.37	234	589.1	1,303.6
3.0	silt	CL	sand	5.03	1,312	559.8	531.5
4.5	silt	CL	sand	5.40	1,562	580.8	563.2
6.0	silt	CH	sand	5.21	841	303.6	325.3
7.5	clay	CH	sand	5.21	400	37.0	34.3
9.0	clay	CL	sand	5.49	456	24.5	23.6
10.5	clay	CL	sand	5.30	298	Bottom of penetrometer push	43.5
12.0	silt	CL	sand	5.43	219		30.6
13.5	sand	CL	sand	6.30	298		35.3
15.0	sand	CL	sand	7.27	182		58.2
16.5	silt	CL	sand	9.13	149		37.2
18.0	silt	CL	sand	9.10	137		41.2
19.5	clay	CL	clay	9.15	164		25.9
21.0	clay	CL	clay	9.13	164		33.6
22.5	sand	CL	silty sand	8.50	285		23.4
24.0	sand	CH	silty sand	8.43	173		94.5
25.5	sand	CH	silty sand	7.76	405		71.1

CL - Sandy clay, silty clay of low to medium plasticity
CH - Inorganic clay of high plasticity

Table 6-3

Summary of Laboratory and Field Data from Borehole 5 and Penetrometer Hole 5

Depth to Top of Interval (ft)	Field Description	Laboratory Classification USCS	Cone Classification	pH	Slurry Resistivity (ohm-ft)	Penetrometer Hole 5 Average Resistivity (ohm-ft)
0.0	silt	CL	sand	7.70	381	0.3
1.5	silt	CL	sand	5.94	1,562	31.5
3.0	silt	CL	sand	5.68	586	16,070.6
4.5	silt	CL	sand	4.47	342	223.8
6.0	silt	CL	sand	4.33	298	197.6
7.5	silt	CH	silty sand	5.00	656	318.2
9.0	silt	CL	silty sand	4.98	443	42.4
10.5	clay	CL	silty sand	5.06	349	31.1
12.0	clay	CH	clay	6.85	228	26.8
13.5	clay	CH	clay	6.65	160	21.5
15.0	clay	CH	clay	8.24	91	16.3
16.5	clay	CH	clay	8.32	80	20.7
18.0	clay	CH	clay	8.31	80	21.5
19.5	sandy clay	CH	silty sand	7.76	99	20.5
21.0	silty sand	CL	silty sand	9.13	205	112.6
22.5	sand	SM	sand	9.03	273	223.2
24.0	clayey sand	CLML	sand	8.93	252	295.0

CL - Sandy clay, silty clay of low to medium plasticity
CH - Inorganic clay of high plasticity
SM - Sand-silt mixtures
CLML - Sandy or silt clay and clayey fine sand

Table 6-4

Summary of Laboratory and Field Data from Borehole 6 and Penetrometer Hole 6

Depth to Top of Interval (ft)	Field Description	Laboratory Classification USCS	Cone Classification	pH	Slurry Resistivity (ohm-ft)	Penetrometer Hole 6 Average Resistivity (ohm-ft)
0.0	sand	CL	sand	7.43	820	295.9
1.5	sand	CL	silty sand	7.80	219	16,365.1
3.0	cavity	cavity	cavity	--	--	37,872.1
4.5	sand	CL	silty sand	4.59	156	1,036.9
6.0	silt	CL	silty sand	4.68	193	129.6
7.5	silt	CL	sand	5.07	656	108.6
9.0	silt	CL	sand	5.30	820	38.8
10.5	clay	CL	silty sand	6.14	410	33.3
12.0	clay	CH	clay	6.12	410	34.6
13.5	clay	CH	clay	6.95	219	27.7
15.0	clay	CH	clay	7.53	234	20.8
16.5	clay	CH	clay	8.29	106	21.6
18.0	sand	CH	sand	8.49	149	21.9
19.5	sand	CH	sand	8.36	109	22.2
21.0	sand	NP	sand	9.41	252	34.6

CL - Sandy clay, silty clay of low to medium plasticity
CH - Inorganic clay of high plasticity
NP - Not plastic

At borehole 6, the CPT classified the upper 12.0 ft (3.6 m) as sand or silty sand; the field classification identified the interval as sand overlying silt and the laboratory classification indicated the soil was non-plastic silty or sandy clay (Table 6-4). The upper part of this boring was in the filled waste pond. A cavity and debris were encountered during drilling indicating that the waste pit had been used as a landfill for solid waste. Below the filled pond, the various classification systems agree closely on the types of soil present.

Except for anomalous soil conditions such as the cementation in the upper part of borehole 1 and the loose, cavity-filled debris and soil in the upper parts of borings 5 and 6, the CPT produced soil classification data that compared well with field and laboratory classification systems. CPT-derived soil data can be combined with data from drilling logs (field classifications) and engineering studies (USCS classifications). CPT can be used to extend trends or patterns of soil types described from field logs or boring samples.

6.5 Soil Resistivity

The resistivity measurements made in the field with the cone penetrometer are influenced by the type of soil, the degree of soil saturation (the position of the water table), and the presence of contamination.

Profiles down-hole for the cone resistivity, laboratory-derived slurry resistivity and slurry pH's are presented in Figure 6-6, 6-7, and 6-8 for all three of the test locations. In each boring the slurry resistivity showed a decreasing trend with depth. The slurry pH showed a consistent increase with depth. The slurry resistivity reflects the concentration of contaminants from closed disposal ponds. The concentration increases with increasing depth indicating the salts wash out of the upper layer and concentrate in the saturated clays and sands below the water table (6 to 8 ft or 1.8 to 2.4 m below the surface). The trend for the wastes (alkaline salt residue) to be leached into the soil units below the water table is also shown by the increase in pH in the deeper soil units.

The data are consistent with the known history of disposal operation in the area. Area soils are typically mildly acid (pH 4.0-6.0) in the test area (U.S. Dept. of Agriculture, 1966; p. 93). Alkaline liquids (treated acidic wastes) disposed in the evaporation ponds and leaching from the waste account for both the increase in salt content (decreased resistivity) and the increased pH (8.0-9.0) observed under the test locations.

The cone penetrometer resistivity tool showed a decrease in resistivity as the concentration of contaminants increased with depth. The cone resistivity also showed a major drop to low and consistent values as the CPT passed below the water table.

In penetrometer hole 2 the cone resistivity shows a sharp decrease as it enters the ground (Figure 6-2) and the unit starts to record. The resistivity remains high down to the top of the water table (8 ft or 2.4 m). Below the water table the probe value is consistently low indicating the presence of the contaminants to the bottom of the investigated interval.

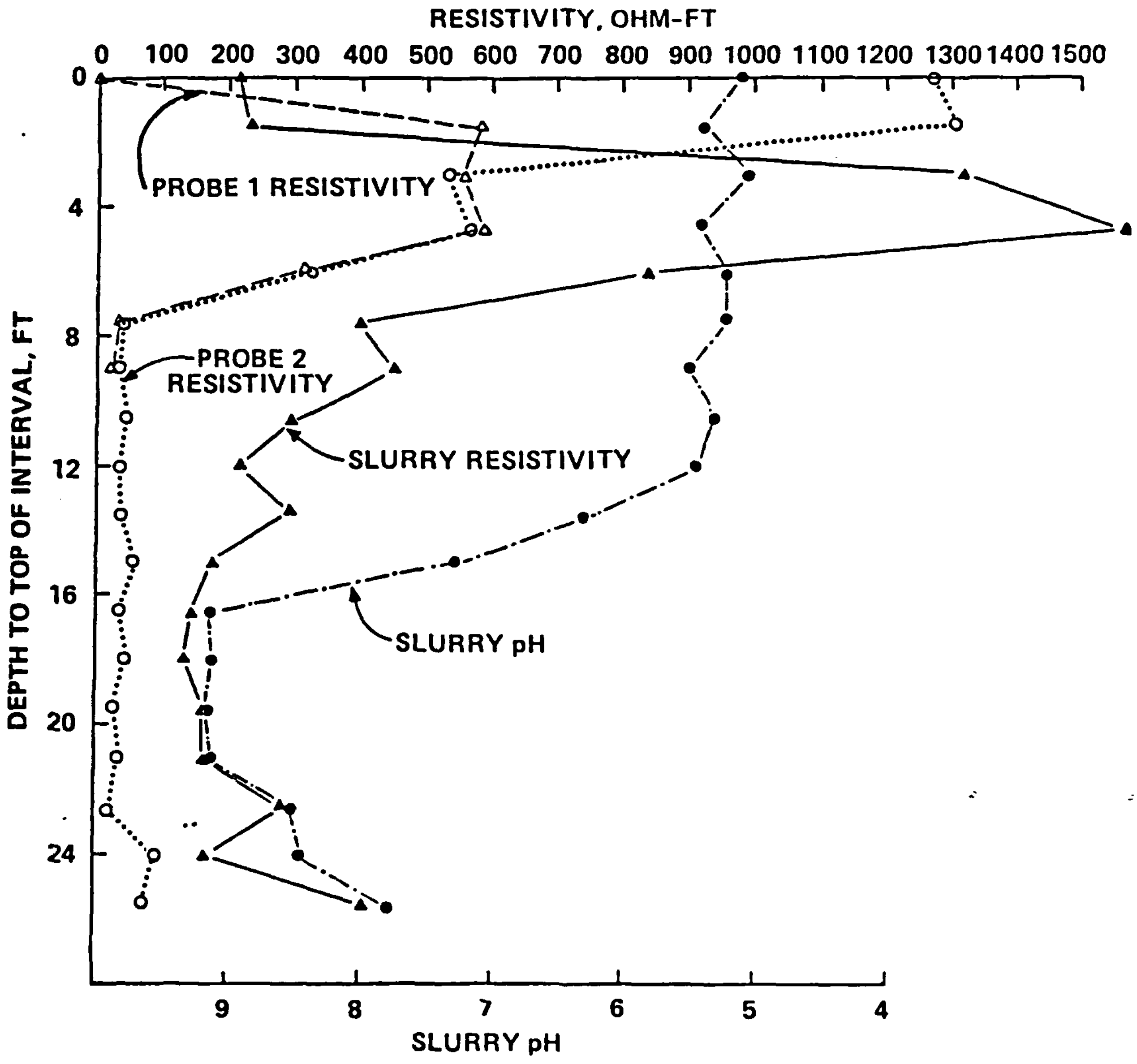


Figure 6-6. Variation of penetrometer (probe) resistivity, slurry resistivity and pH with depth at borehole 1.

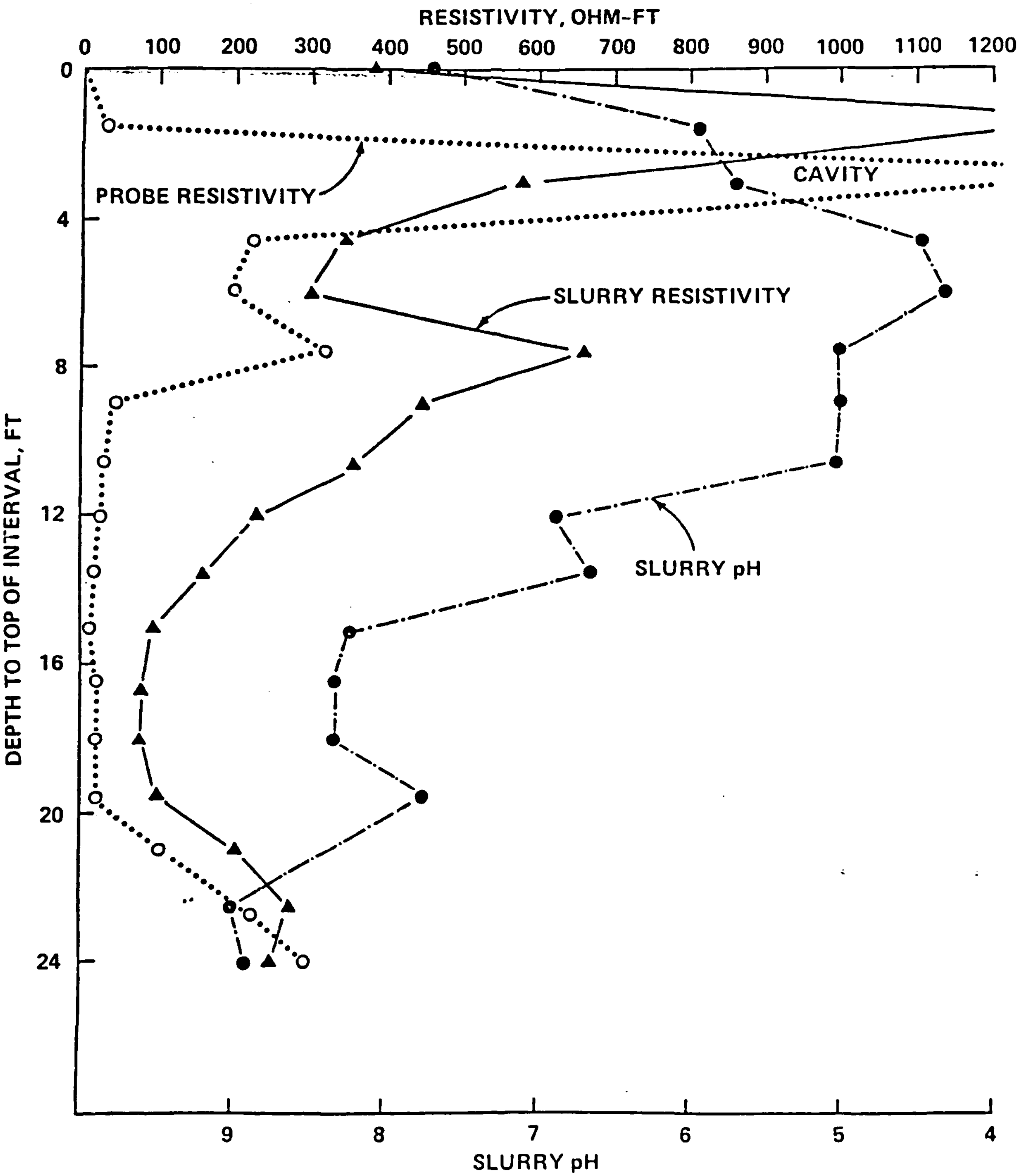


Figure 6-7. Variation of penetrometer (probe) resistivity, slurry resistivity and pH with depth at borehole 5.

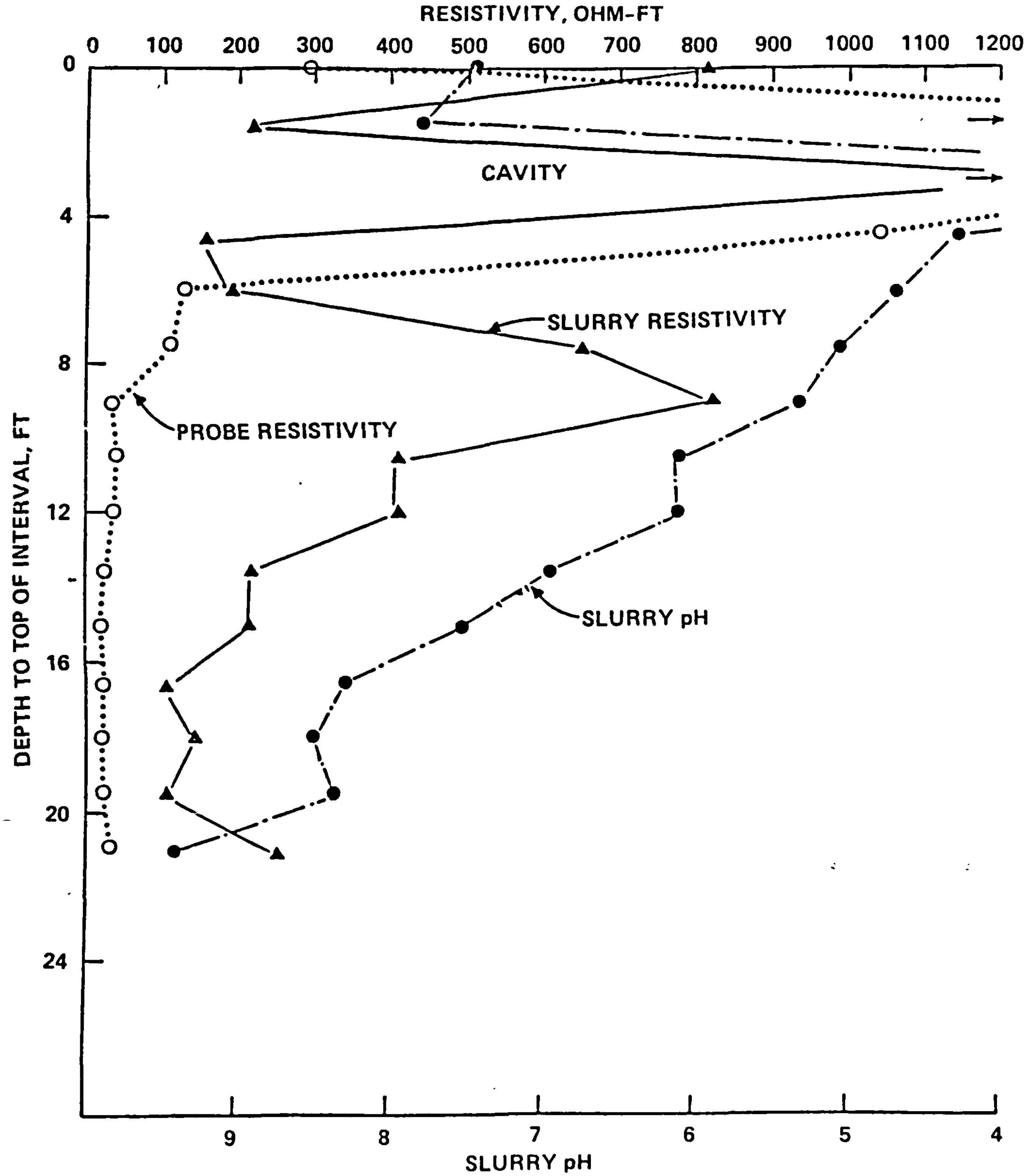


Figure 6-8. Variation of penetrometer (probe) resistivity, slurry resistivity and pH with depth at borehole 6.

In penetrometer hole 5 (Figure 6-3), the cone resistivity shows several extreme fluctuations in the top 3 to 5 ft (0.9 to 1.5 m). The unusually low resistivities indicate a conductive layer, probably a salt layer in the evaporation pond. The resistivity drops to low and consistent values below the water table.

In penetrometer hole 6, fluctuations in the resistivity readings at the 4- to 5-ft (1.2- to 1.5-m) depth indicate that the conductive layers noted in penetrometer hole 5 are also present in hole 6 (Figure 6-4). The resistivity decreases to low and consistent values below the water table.

The resistivity values noted below the water table are generally lower than would be expected for clean sands and clays indicating contaminants were present below the water table at all three CPT locations. In the area of borehole 6 and penetrometer hole 6, the concentration of contaminants decreased from 20 to 24 ft (6 m to 7.3 m) depth, as shown by the slurry resistivity (Figure 6-8). The CPT resistivity module showed a similar decrease in resistivity.

The CPT resistivity unit was able to locate conductive layers associated with contaminants above the water table. The resistivity unit also responded to the top of the water table with a drop in resistivity. Resistivity levels below the water table also indicate the presence of contaminants.

7.0 CONCLUSIONS

In general, the cone penetrometer system performed well in the proof of concept testing at LAAP. The penetrometer hardware, instrumentation, virtual realtime data processing, and plotting routines all functioned as planned. Also, zones known to be contaminated were detected by the resistivity module as an anomalously low resistivity interval in the soil profile.

The cone penetrometer test results were in reasonably good agreement with the drilling, sampling, and laboratory testing results, with one exception. The soil classification calculated by the penetrometer data processing code did not agree with the laboratory soil classification in the upper 10 to 15 ft (3 to 4.6 m) of the soil profile at LAAP. This interval contains well cemented sand and clay layers, and the current classification algorithm did not correctly identify these materials according to soil type. The point and sleeve resistance measurements provide detailed data on the depth and thickness of soil layers even when they are cemented, and these data are useful in determining site stratigraphy. Experience at this site confirms the need for a few well-placed sample borings (whose placement can be optimized using the penetrometer data) to provide the physical evidence needed for an accurate interpretation of subsurface conditions. This approach is consistent with the HWS assessment strategy described earlier in this report.

All of the experimental penetrometer holes at LAAP were grouted closed. Each hole required approximately 10% more than the nominal volume for the probe hole. In tests conducted at the WES, grouted penetrometer holes were excavated and examined. The penetrometer grouting system produced continuous grout columns with no gaps.

8.0 RECOMMENDATIONS

8.1 Current Developments

The current WES cone penetrometer development program includes interim refinements to the grouting cone, resistivity module, and the data processing software. It is recommended that the classification algorithm be studied for possible modifications to correctly account for cemented and/or heavily overconsolidated materials. It may be possible to effect this improvement by integrating resistivity data with the current computer algorithm which now uses only point and sleeve penetration resistance values to calculate soil type. It is also recommended that field testing be extended to acquire an adequate data base to optimize system performance in a variety of geologic settings. Spare cones and resistivity modules are now under construction in anticipation of more intensive testing.

8.2 Future Developments

There is continuing need for a penetrometer sensor(s) which can both detect and identify a broad spectrum of insitu contaminants. The WES is currently developing an innovative cone penetrometer which utilizes fiber optics and spectral analysis technology for contaminant detection insitu. Laboratory tests of spectral analysis through fiber optics have demonstrated a capability to detect petroleum, oil, and lubricants (POL) in soil. It is recommended that further research be directed to the use of fluorescence, reflection and absorption techniques to detect and identify POL and other contaminants of concern. It is further recommended that a prototype fiberoptic cone be constructed for proof of concept field testing in FY 89.

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APPENDIX A

LABORATORY SOIL TESTING PROCEDURES

The soil testing was done to permit the classification of the soil under the criteria used in the Unified Soil Classification System (USCS). The USCS (Water and Power Resources Service, 1974) requires grain-size analysis and the determination of Atterberg limits (liquid and plastic limits).

Grain Size Analysis. Grain-size analysis is the procedure in which the proportion of soil of each grain size in a given soil is determined. The grain size determination for this study was done by sieve analysis and was restricted to material retained on US Standard No. 200 sieve. In the USCS classification, the fraction passing the No. 200 sieve is considered clay and no further grain size analysis is needed on this fraction.

The general procedure is given in detail in the Laboratory Soils Testing (Office of the Chief of Engineers, 1970) Appendix V (EM 1110-2-1906) and the specific steps used in tested soils are as follows:

(a) The samples were oven-dried at $110^{\circ} \pm 5^{\circ}C$ and allowed to cool and were weighed The samples were all less than 500 grams and were weighed to the nearest 0.1 grams.

(b) After initial drying, the samples were thoroughly broken up using the fingers and a rubber-covered mortar and pestle. The disaggregated sample was split into a sample equal or greater than 50 grams.

(c) The samples were mixtures of sand and plastic fines and it was necessary to soak the samples in water for 2 to 24 hr prior to sieving.

(d) Each sample was washed through a combined No. 4 and No. 200 sieve. Material passing the 200 mesh sieve was discarded.

(e) After drying a second time the samples were reweighed and sieved through a sieve stack containing US Standard Sieve No. 4-1/2, 5, 10, 16, 20, 30, 40, 50, 70, 100, 140, and 200. The nest of sieves was agitated in a mechanical shaker for 10 min or until no appreciable change in the proportion of material on each sieve changed.

(f) Data were tabulated and grain-size curves were prepared.

Atterberg Limits (liquid and plastic limits). Atterberg limits are the water contents (as a weight fraction of the soil) that define the changes in the consistency for a fine-grained soil. The liquid limit is the water content at which two halves of a soil pat separated by a groove prepared with a standard grooving tool will close along a distance of 0.5 in. (1.25 cm) under an impact of 25 blows in a standard liquid limit device. The plastic limit of a soil is the water content at which the soil just begins to crumble into pieces when rolled into a thread 0.125 in. (0.32 cm) in diameter.

The liquid and plastic limits were determined using the general procedure outlined in Laboratory Soils Testing, Appendix III (Office of the Chief of Engineers, 1970). Specific procedures involved the following steps:

(a) Samples were used at their natural water content.

(b) Samples containing particles over a No. 40 sieve were washed to remove the greater than No. 40 sieve fraction.

(c) The liquid limit (25 blows) was interpolated from a straight line plot of blow count versus water content.

(d) The plastic limit was determined from a sample prepared for the liquid limit test.

(e) All plastic limit water content determinations are made in duplicate and averaged.

(f) The plasticity index is calculated as the liquid limit minus the plastic limit.

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APPENDIX B

LABORATORY CHEMICAL METHODS OF SOIL ANALYSIS

Measurements of resistivity (or conductance) and of soil pH were undertaken on samples recovered from the test site. Measurements of resistivity were made to confirm soil resistivity trends observed at the test site. Soil pH measurements were made to trace the movement of alkaline waste out of the closed evaporation ponds.

Soil conductivity is the reciprocal of soil resistivity. The soil conductance was measured using the general method outlined for soil-water mixtures (Dewis and Freitas, 1970, p 85). The general procedure presented was employed using the following specific steps:

(a) The samples were oven dried at $105^{\circ} \pm 5^{\circ}C$ to a constant weight.

(b) A 10-gram aliquot of dried soil was mixed with 50 ml of distilled deionized water. The samples were shaken for 60 minutes on a reciprocating shaker.

(c) The samples were allowed to settle for approximately two hours and then centrifuged.

(d) Conductivity was measured using a dipping cell that was calibrated against a standard potassium chloride solution.

Soil pH was measured using the technique outlined in Dewis and Freitas (1970, p 65). The specific procedure involved the following steps:

(a) The soil was dried at $105^{\circ} \pm 5^{\circ}C$ to a constant weight.

(b) A 10-gram aliquot of dried soil was mixed with 50 ml of 0.01 m calcium chloride solution. The soil suspension was shaken for 15 minutes on a reciprocating shaker. After shaking the suspension was allowed to settle for 30 to 60 minutes.

(c) The pH was measured using a calibrated glass electrode and reference electrode immersed approximately 1 in. (2.5 cm) into the top of the settled suspension.

REFERENCE

Dewis, J. and Freitas, F. 1970. Physical and Chemical Methods of Soil and Water Analysis." Soils Bull. 10, Food and Agricultural Organization of the United Nations, Rome, 275 pp.

APPENDIX C

CONE PENETROMETER COST/BENEFIT ANALYSIS

Conventional Practice is to use drilling, sampling, and laboratory sample testing to characterize subsurface conditions and to locate and identify any hazardous wastes present. A common practice for exploratory borings in soil is to use a hollow stem auger with split spoon sampling of 50 percent or less of the material drilled. Based on recent WES experience, the average cost for this kind of drilling and 50 percent sampling is $18/ft (in 1986 dollars) for holes to 50 ft in depth. Other drilling and sampling methods are more expensive, for example rotary drilling and 50 percent sampling with 3-in. diameter Shelby tubes averages about $34/ft. To these costs must be added the cost of conventional soil material property tests, such as soil classification, grain density, Atterburg limits, grain size distribution, etc., or other special laboratory testing of the samples. Conventional soil property testing costs about $25/ft of sample, or more. Hence, the least average cost of drilling, sampling, and conventional material property testing (exclusive of any chemical testing) is on the order of $18/ft + 0.50 ($25) or $30.50/ft with a hollow stem auger and split spoon sampling of 50 percent of the material drilled. Intermittent (skip) sampling of the soil media may not provide the information needed to either accurately characterize subsurface conditions or properly locate monitoring wells, as discussed earlier in this report. However, "skip" sampling is a common practice, and is used as the basis of cost comparisons for this reason.

Cone penetrometers can provide a detailed and continuous record of site stratigraphy and soil type. Based on recent contacts with firms providing such services, the basic cost for standard electric CPT cones is about $6/ft of penetration, in addition to transport to the site and a nominal setup charge. The WES is developing penetrometer sensors which should provide the capability to detect moderate to high concentrations of contaminants in situ. With a sensor of this kind, cone penetrometers could be used to detect and delineate contaminated zones in the soil media. It is important to consider any operational savings which could be realized with a multi-sensor penetrometer capable of both site characterization and detection/delineation of contaminants. The resistivity measurement module described in this report is capable of taking continuous measurements at the standard rate of penetration, i.e., 2 cm/sec. Other sensors under consideration could require pauses in penetration, for example to acquire gas/fluid samples for measurement in the probe. Pausing to take discrete measurements or physical samples would increase time, hence cost, and a worst case estimate of the increase would be a doubling in cost to $12 rather than $6/ft. Even if using a cone of this kind were to result in a net doubling of cost per foot of penetration (i.e., $12/ft), the savings would be $\frac{\$30-12}{30}$ X 100 or 60 percent less than the cost of augering and 50 percent split spoon sampling to characterize soil conditions. The approach of using penetrometers to detect/delineate contaminated zones could provide much greater savings if the costs of conventional laboratory chemical analyses are considered. For example, conventional laboratory analyses for organic contaminants typically cost $1,000/sample, so detection and delineation of contaminated zones using drilling/sampling/sample analysis procedures can be

very expensive. The direct physical evidence provided by drilling/sampling/ sample analysis is an essential part of HWS assessments, but very substantial savings could be realized if this procedure was limited to confirming findings obtained by more economical means, i.e., as from cone penetrometers equipped with contaminant sensors.

An example may serve to illustrate the kind of flexibility and savings this approach could provide. Some attention has recently been given to the problem of locating an anomaly (for example, a cavity or polluted zone) in either a rectangular or square field of search using random or systematic drilling techniques. An intuitive reaction is that a systematic approach is better, and this has been proven theoretically (Franklin, et al., 1981; Benson, et al., 1982). An example of a rectangular site containing a target (hazardous waste source), which has an area equal to 10 percent of the total site area, is shown in Figure C-1 (Benson, et al., 1982). Statistically-derived probability curves for the borings required to locate this target using random or systematic techniques are shown in Figure C-2 (Benson, et al., 1982). Results of earlier WES studies for a square site having a 10 percent target area are shown in Figure C-3, and these results exhibit very similar trends (Franklin, et al., 1981). Referring to Figure C-3, it is seen that 16 square-gridded borings are needed to locate the target with virtually 100 percent confidence. Approximately twice as many random borings would be required to produce a similar level of confidence, so a random approach is not likely to prove cost effective. Other aspects of optimal search plans and theory are available from the literature (Savinski, 1965; Singer and Wuckman, 1969; Allais, 1951; Engle, 1957) but will not be discussed further herein.

If it is assumed that the hypothetical (square grid) site investigation requires 16 exploratory borings to 50 ft in depth, then some cost comparisons can be derived as follows:

Optimal Site Investigation to 50 Feet in Depth

Case I	Case II
Electric Cone Penetrometer with Proposed HW Sensor (16 Penetrations)	Conventional Drilling/Sampling/Soil Property Testing Using Auger and Split Spoon (16 Borings)
Exploration Cost (16x50x$12) - $ 9,600	16x50x$30.50 - $24,400
+ Three Confirmation Borings - 4,575 (3x50x$30.50) - $14,175	

Providing that the above cost estimates for the soils exploration are accurate, as they are believed to be, then the Case I approach using penetrometers would clearly be preferred. The optimum location and depths of verification sampling would be known in advance. Site coverage could also be enhanced by additional penetrations, for example to better define a pollution plume, so that the Case I approach is inherently more flexible and cost-effective than using only exploratory drilliung/sampling (Case II). Also, the position of monitoring wells could be optimized by location and depth to set well screens.

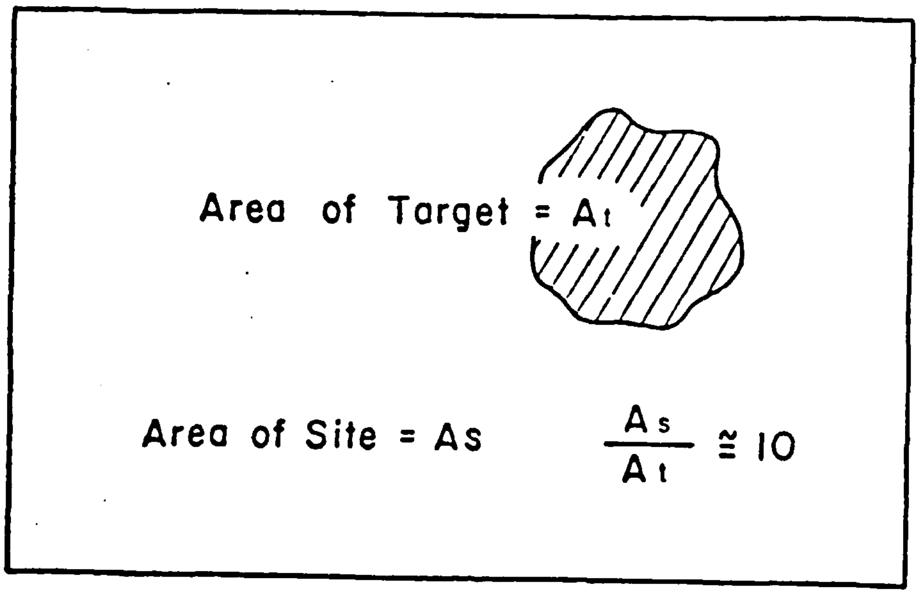

Figure C-1. Ratio of overall site area to target area is often large. Target area may represent a plume or burial site. Smaller targets become more difficult to find. (after Benson, et al., 1982)

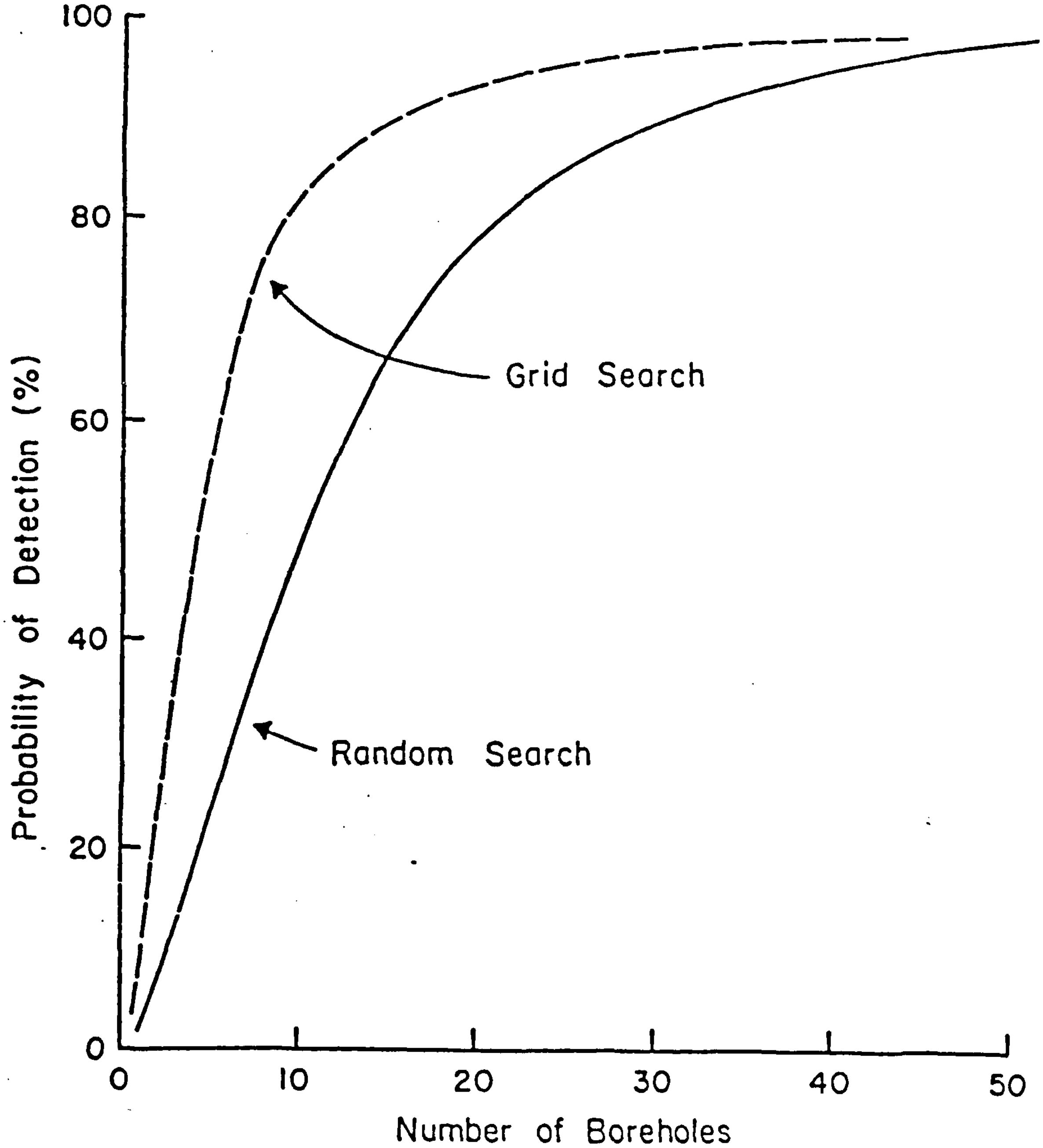

Figure C-2. Probability of detecting a target using a rectangular grid and randomly located borings. Data for A_s/A_t Ratio of 10. (after Benson, et al., 1982)

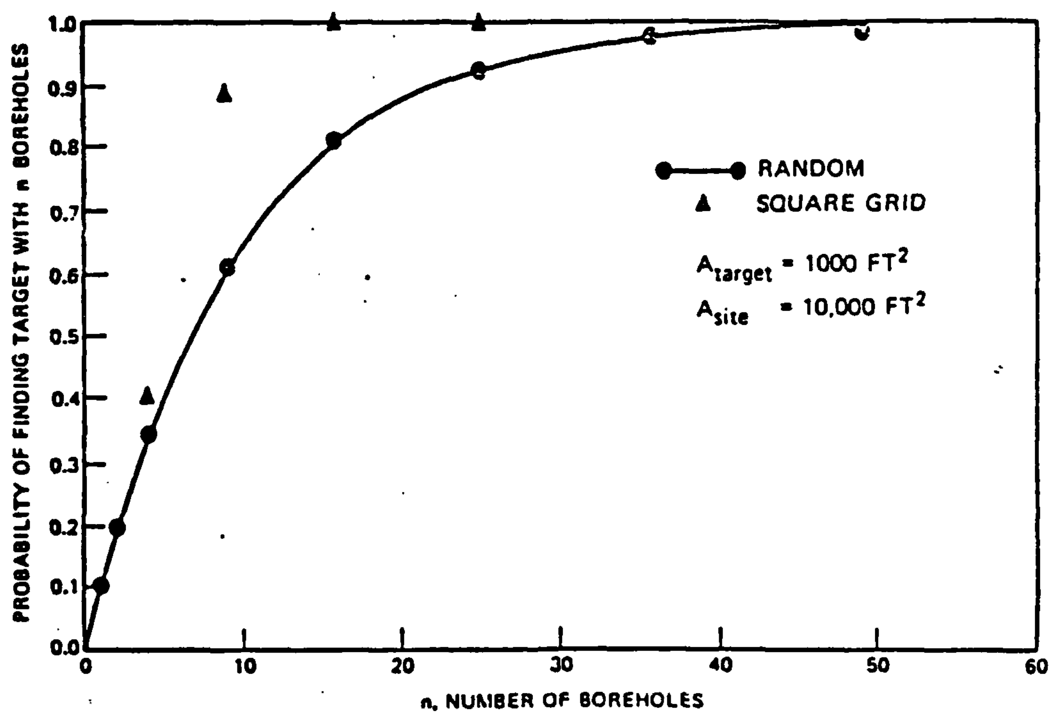

Figure C-3. Probability of finding a target with borings in a square grid pattern or randomly located borings. $A_s/A_t = 10$ (after Franklin, et al., 1981)

The three confirmation borings proposed for Case I above would also serve to correlate lab-tested material properties with cone penetrometer data so that little, if any sacrifice in the accuracy of soil property determinations need be expected. The above figures do not include the cost of conventional laboratory chemical analyses of samples, which are usually a major expense in HWS assessments. However, using only three borings to confirm findings and identify contaminants is clearly more economical than using 16 borings with drilling/sampling/sample analysis to achieve the same ends. Using the example cited, the cost of laboratory chemical analyses could be reduced to 3/16 or approximately 1/5 of the usual cost.

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